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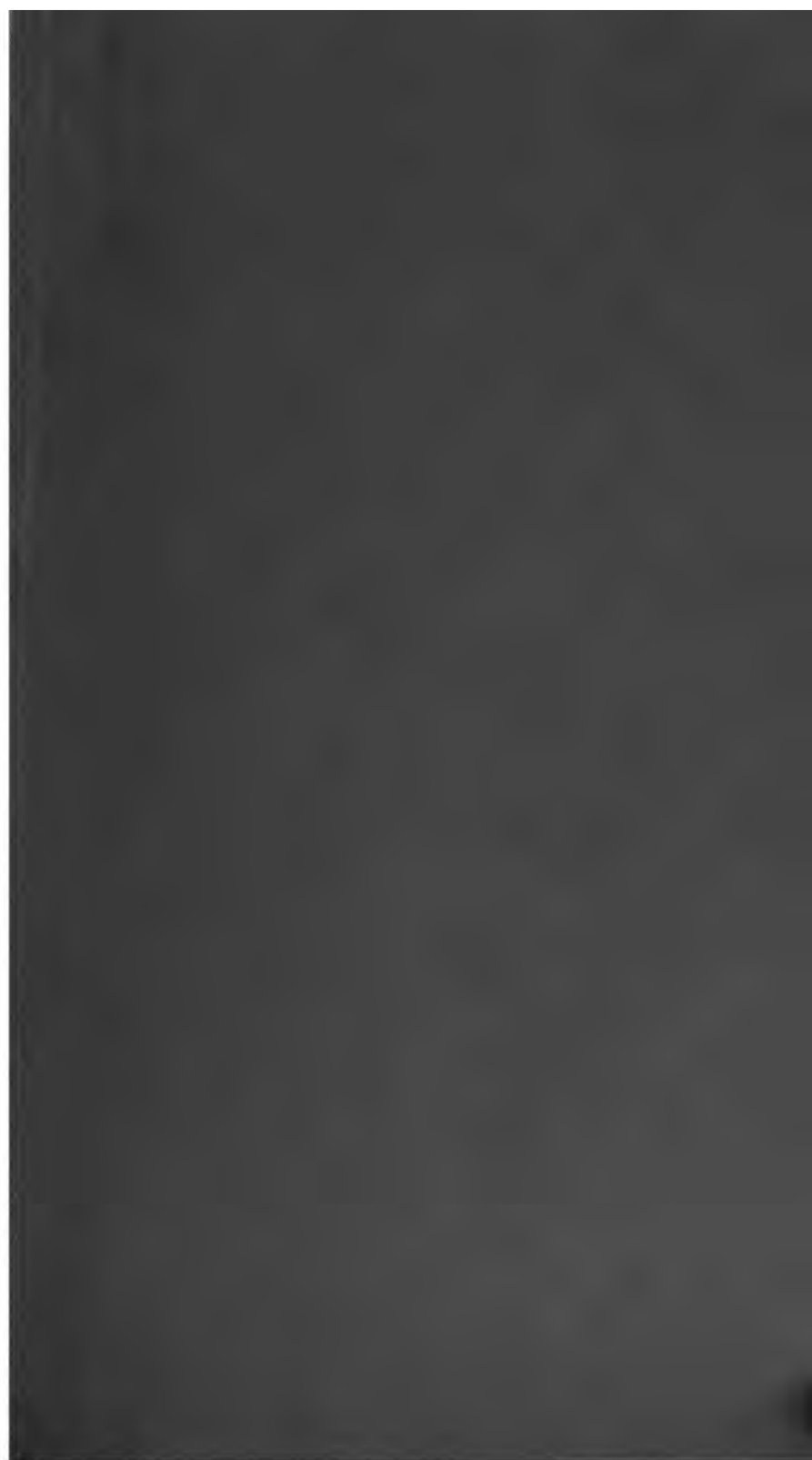
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# LIVE-LOAD STRESSES

IN

## RAILWAY BRIDGES

WITH

### FORMULAS AND TABLES

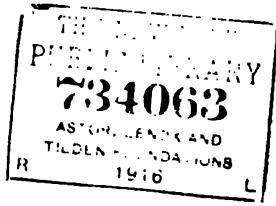
BY

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## PREFACE

STRESSES caused by moving concentrated loads are treated in this book by the combined use of influence lines and algebraic methods. The influence line is connected by this treatment with tables of moment sums and load sums in a new and entirely practical manner.

The heart of the text is contained in equations (7) and (8). These give an easy and exact solution of the maximum live-load stresses in any structure whose influence lines can be drawn, replacing, for the more complicated structures, such as cantilever and swing bridges, arches, etc., the old method of placing the wheel loading by trial and scaling the influence-line ordinates under the loads.

A second feature of the text is the application of equations (7) and (8) to the simpler structures, such as girder bridges (with and without panels), pier reactions, and Pratt trusses (with inclined and horizontal chords), in which these equations are transformed and simplified to meet the requirements of these ordinary cases. This leads to a series of simple formulas to meet the needs of every-day designing. To illustrate the application of these formulas, fully worked-out examples are given.

The text is supplemented by a very complete set of tables, the usefulness of which is at once apparent. The greater part of the matter in these tables is new. A table similar to Table 3 was made by Mr. Josiah Gibson, C.E., and published in the *Engineering News*, June 21, 1906; and a table similar to Table 11 is given by Mr. J. P. J. Williams in the *Engineering News* of Oct. 1, 1914. Tables similar to Tables 6, 8, and 9 are found in the "Structural Engineers' Handbook" by Dean Milo S. Ketchum and in the "Design of Steel Bridges" by Mr. F. C. Kunz.

The author wishes to acknowledge his indebtedness to the American Bridge Company for material assistance, and in particular to Mr. O. E. Hovey, Assistant Chief Engineer of this company, for his encouragement and help. The author also desires to acknowledge the valuable suggestions made in the revision of the original text by Professor F. H. Constant, of the Civil Engineering Department of Princeton. To Professor William H. Burr of Columbia University, the writer is permanently indebted for the logical and thorough instruction received from him as a student.

G. E. B.

PRINCETON UNIVERSITY  
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# LIVE-LOAD STRESSES

## ARTICLE I.

### INFLUENCE LINES. DEFINITION AND USES.

INFLUENCE lines are useful in determining the position of live load on a bridge to produce maximum effect. They offer also a convenient method of deriving general algebraic formulas for stresses and rules for maximum when the general relations between influence lines and algebraic formulas are once understood; and in the case of the more complex problems of skew bridges, arches, cantilever bridges, etc., the influence lines themselves serve as a most direct method for the determination of the maximum live-load stresses.

An influence line may be defined as a line showing the variation in any function caused by a single *unit* load as it moves across the bridge. Vertical loads only will be considered. The function may be a reaction, bending moment, shear, stress, deflection, or any quantity whatsoever at a given part of a bridge, provided that its value is a function of the position of the unit load on the bridge.

Refer to Fig. 1a. Consider the span  $AB$ , and let  $Z$  be any function at the fixed position  $C$  on the span  $L$ . If the load unity moves across the span  $AB$  and the value of  $Z$  be calculated for each position of the unit load and its value  $z$  plotted below the corresponding position of this load as an ordinate from a horizontal base line, the locus of the plotted points will be the influence line for  $Z$ . For example, if  $Z$  be the bending moment at the fixed section  $C$  in a beam of span  $L$ , the influence line will be as shown in Fig. 1b. In plotting influence lines, ordinates repre-

representing positive quantities are plotted above the base line; and negative, below. In case the influence line consists of several straight segments, it is necessary to determine the value of the ordinates only where the influence line has a change of direction; i.e., at the *salient points*. For example,

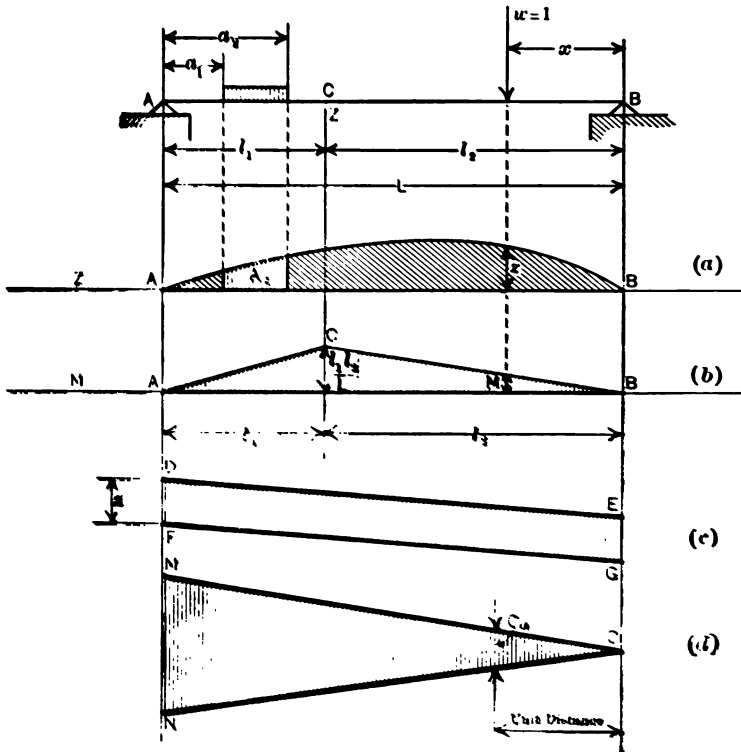


FIG. 1.

the points A, C, and B are the salient points of the influence line in Fig. 1b.

The value of  $Z$  caused by a single load  $w$  is equal to  $wz$ , if  $z$  is the influence ordinate below  $w$ . The value of  $Z$  caused by a series of loads  $w_1, w_2, w_3$ , etc., is

$$Z = w_1 z_1 + w_2 z_2 + w_3 z_3 + \dots = \sum w_i z_i \dots (1)$$

where  $z_1, z_2, z_3$ , etc., are the influence ordinates below the corresponding loads. It will be convenient to speak of such a quantity as  $wz$  as an *ordinate-load product*.

Formula (1) therefore may be expressed thus:

$Z = \text{Sum of ordinate-load products.}$

The area between the influence line and the base line is called the *influence area*. It may be shown that the value of  $Z$  caused by a uniform load on the bridge is proportional to the area  $A_z$  of the influence line between the ordinates at the extremities of the uniform load. If the uniform load in Fig. 1a has an intensity of  $q$  per unit of length, the load in the length  $dx$  equals  $q dx$ , and the influence of this elementary load on the value of  $Z$  is  $zq dx$ , where  $z$  is the influence ordinate below  $q dx$ . Summing up for the length of the uniform load,

$$Z = q \sum_{a_1}^{a_2} z dx = qA_z \quad . . . . . (2)$$

If a series of equal loads  $w$  is on the span, the value of  $Z$  is

$$Z = \Sigma wz = w \Sigma z \quad . . . . . (3)$$

If a series of unequal loads,  $w_1, w_2$ , etc., is multiplied by the corresponding ordinates of an influence line or a portion of an influence line which has a constant ordinate  $z$ , as in Fig. 1c, the value of  $Z$  is

$$Z = z(w_1 + w_2 + \dots) = z \Sigma w = zW \quad . . . (4)$$

where  $W$  equals the sum of these loads.

If a series of unequal loads is multiplied by the corresponding ordinates of an influence line or a portion of an influence line consisting of two diverging lines, as shown in Fig. 1d, the value of  $Z$ , or the sum of the ordinate load products, and the rate at which  $Z$  varies as the loading advances, are given by the two theorems that follow. The *slope* of a line is defined at the beginning of Art. 2.

*Theorem I.*

The sum of the ordinate-load products between two diverging lines equals the difference between the *slopes* of the two lines multiplied by the sum of the moments of the loads about the intersection of these lines.

In symbols, this is stated as

$$Z = C_a M_a \dots \dots \dots (5)$$

*Theorem II.*

The rate at which the sum of the ordinate-load products between the two diverging lines increases as the loading moves away from the intersection of these lines equals the difference between the *slopes* of the two lines multiplied by the sum of the loads.

In symbols, this is stated as

$$\frac{dZ}{dx} = C_a W_a = \frac{d(C_a M_a)}{dx} = C_a \frac{dM_a}{dx} \dots \dots \dots (5a)$$

The proofs of these theorems follow in the next article.

## ARTICLE II.

### SUM AND RATE OF VARIATION OF ORDINATE-LOAD PRODUCTS BETWEEN THE TWO DIVERGING LINES.

CONSIDER the diverging lines  $DAB$  and  $AC$  in Fig. 2.  
Use the following notation:

$w$  = any vertical load.

$z$  = ordinate below  $w$  in the angle  $BAC$ .

$Z = \Sigma w_n z_n$  = sum of ordinate-load products.

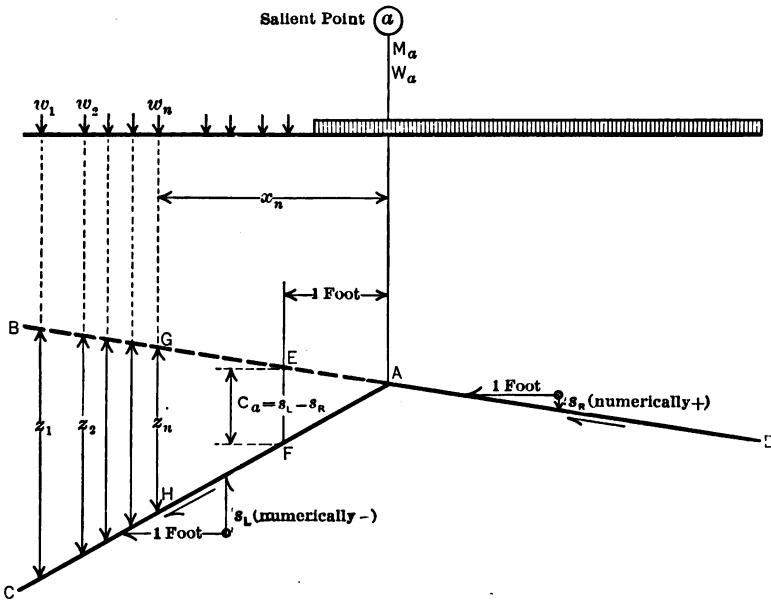


FIG. 2.

$M_a = \Sigma w_n x_n$  = moment sum of all loads to left of  $Aa$   
about A.

$W_a = \Sigma w_n$  = load sum of all loads to left of  $Aa$ .

$s_R$  = slope of line  $DA$  = tangent of angle which  $DA$   
makes with the horizontal.

$s_L$  = slope of line  $AC$  = tangent of angle which  $AC$  makes with the horizontal.

$C_a = \frac{z_n}{x_n} = (s_L - s_R)$  = length of ordinate unit distance from  $A$ .

Slopes are counted numerically positive when upward to the left. The sign of  $C_a$  (called the coefficient at salient point  $A$ ) is, accordingly, negative when  $AC$  diverges below  $DA$  produced to the left of  $A$ . The value of  $C_a$  may be determined graphically as  $\frac{z_n}{x_n}$  or it may be figured algebraically as  $(s_L - s_R)$ .

*Proof of Theorem I, or that  $Z = C_a M_a$ .*

Consider the load  $w_n$  distant  $x_n$  from the salient point  $a$ . By the similar triangles  $AEF$  and  $AGH$ ,

$$\frac{C_a}{1.00} = \frac{z_n}{x_n}, \text{ or } z_n = C_a x_n.$$

Therefore,

$$w_n z_n = C_a w_n x_n. \quad \dots \dots \dots (A)$$

Summing up all of the ordinate-load products,

$$Z = \Sigma w_n z_n = C_a \Sigma w_n x_n = C_a M_a. \quad \dots \dots \dots (5)$$

*Proof of Theorem II, or that  $\frac{dZ}{dx} = C_a W_a$ .*

From equation (A) above, the increase in the ordinate-load product  $w_n z_n$  for an advance  $dx_n$  of the load is

$$w_n dz_n = C_a w_n dx_n.$$

Summing up the increases of all the ordinate-load products and noting that  $dx$  is the same for all loads,

$$dZ = \Sigma w_n dz_n = C_a dx \cdot \Sigma w_n = C_a W_a \cdot dx.$$

Dividing by  $dx$ ,  $\frac{dZ}{dx} = C_a W_a = \frac{d(C_a M_a)}{dx} = \frac{C_a dM_a}{dx}. \quad \dots \dots \dots (5a)$

### ARTICLE III.

#### SUM AND RATE OF VARIATION OF ORDINATE-LOAD PRODUCTS FOR ANY INFLUENCE LINE. POSITION OF LOADING FOR MAXIMUM LIVE-LOAD STRESS.

AN influence line of a general type is shown in Fig. 3, this one in particular being for the member  $U_3L_4$  of the

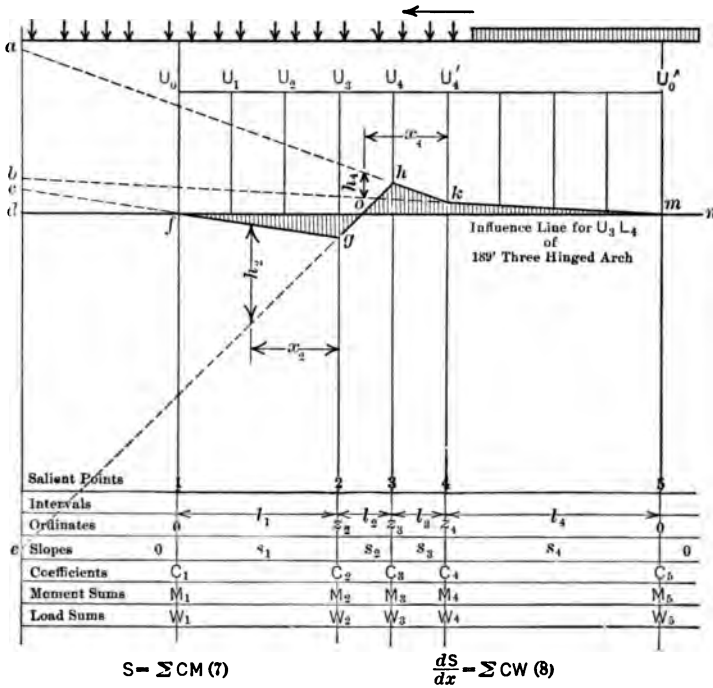


FIG. 3.

arch shown in Fig. 15. It is assumed that the ordinates at all salient points and the intervals between these points are known. Ordinates and slopes are counted positive or negative as already defined. The slope of any segment of the



influence line equals the ordinate at the left minus the ordinate at the right end of this segment divided by the corresponding interval. The *coefficient C at any salient point* equals the slope of the segment at the left minus the slope of the segment at the right of this point. The subtractions in each case are made algebraically.

It should be remembered, as has already been pointed out in Art. 2, that the value of any coefficient *C* may also be measured graphically from an influence line which has been drawn to scale. For example, in Fig. 3 the value of

the coefficient  $C_2 = \frac{h_2}{x_2}$  and  $C_4 = \frac{h_4}{x_4}$ .

The algebraic calculation of the coefficients at all salient points of the influence line in Fig. 3 is given below. If it be assumed that this influence line has been drawn to scale, the signs of the numerical values of the slopes and coefficients will be as given in the parentheses.

$$\begin{array}{ll} s_1 = \frac{0 - z_2}{l_1} (+) & C_1 = 0 - s_1 (-) \\ s_2 = \frac{z_2 - z_3}{l_2} (-) & C_2 = s_1 - s_2 (+) \\ s_3 = \frac{z_3 - z_4}{l_3} (+) & C_3 = s_2 - s_3 (-) \\ s_4 = \frac{z_4 - 0}{l_4} (+) & C_4 = s_3 - s_4 (+) \\ & C_5 = s_4 - 0 (+) \end{array}$$

A numerical evaluation of the slopes and coefficients for this influence line is given in Fig. 15 of Art. 8, which the reader should check in order to understand completely the method of procedure. These coefficients should also be checked by the graphical method as already explained.

For example, in Fig. 15 the value of  $C_2 = \frac{2.59}{30} = .0863$ .

It will be noted in the algebraic calculation of the coefficients *C* at all salient points that each slope enters once

s positive and once as negative. Therefore the sum of all coefficients equals zero.

$$\Sigma C = 0. \dots \dots \dots (6)$$

This formula serves as a check on the values of the coefficients which have been determined either by calculation or by graphical measurement.

The general formulas for the sum of the ordinate-load products for any influence line (viz., with several salient points such as the one shown in Fig. 3) may be arrived at by considering the two contiguous sloping sides of the influence line meeting at each salient point as two diverging lines. The entire influence line is thus made up of pairs of diverging lines (see Fig. 3) to each pair of which formula (5) may be directly applied. Thus in Fig. 3,

Ordinate-load products in	$\frac{dfc}{\dots}$	$= C_1 M_1$	(-)
"	"	$\frac{cge}{\dots}$	(+)
"	"	$\frac{eha}{\dots}$	(-)
"	"	$\frac{akb}{\dots}$	(+)
"	"	$\frac{bmd}{\dots}$	(+)

The signs of the  $CM$ 's are + or - according to the signs of the coefficients, for the  $M$ 's are always positive. Summing up the above equations and observing that the ordinate-load products cancel one another except between the influence line  $fghkm$  and its base line  $fom$ , it follows that the sum of the ordinate-load products for the influence line, or the live-load stress, is

$$S = C_1 M_1 + C_2 M_2 + \dots = \Sigma CM. \dots \dots (7)$$

The letter  $S$  represents in general any stress or sum of ordinate-load products for any influence line, while  $Z$  stands for the sum of ordinate-load products for any geometrical figure.

The rate at which  $S$  varies as the load advances a distance  $dx$  equals

$$\frac{dS}{dx} = \frac{d(C_1M_1)}{dx} + \frac{d(C_2M_2)}{dx} + \text{Etc.}$$

But by formula (5a) this becomes

$$\frac{dS}{dx} = C_1W_1 + C_2W_2 + \dots = \Sigma CW. \dots (8)$$

$W_1, W_2$ , etc., = sum of all of the loads to the left of points 1, 2, etc., respectively, whether on the span or not

$M_1, M_2$ , etc., = moment of the same loads about points 1, 2, etc., respectively, whether on the span or not.

The above formulas (6), (7), and (8) apply equally well when the loading is headed from left to right instead of from right to left, the latter being the more usual way. In applying these formulas, however, it will save confusion not to reverse the loading, but to turn the influence line end for end, for this operation changes neither the values nor the signs of the coefficients  $C$ .

The stress  $S = \Sigma CM$  is related to its derivative  $\frac{dS}{dx} = \Sigma CW$  in the same way that any function is related to its derivative. Thus, if the value of  $\frac{dS}{dx}$  passes through zero as the loading advances, the stress itself may have reached any one of four conditions; namely,

1. Numerically maximum positive value.
2. " minimum " "
3. " maximum negative "
4. " minimum " "

In practice it is desirable to find the positions of loading to satisfy the first and third conditions. This may be done by proceeding as directed below. It is assumed in stating the following rules that the live load is advancing from right to left. In case the live load advances from left to right, the wheel will be tried first to the left and

then to the right of a salient point. In other words,  $dx$  is always an increment in the same direction as the loading advances.

*Rule 1.*—To determine the position of loading to give a maximum positive stress, place the live load on the part of the bridge corresponding to the positive portion of the influence line. Try a wheel first immediately to the right of a salient point that has a *negative* coefficient and then just to the left of this point. Calculate the value of  $\frac{dS}{dx} = \Sigma CW$  for each of these successive positions of loading. If the sign of  $\frac{dS}{dx}$  changes from  $+$  to  $-$ , a position of loading for maximum positive stress is determined.

*Rule 2.*—To determine the position of loading to give a numerically maximum negative stress, place the live load on that part of the bridge corresponding to the negative portion of the influence line. Try a wheel first immediately to the right of a salient point that has a *positive* coefficient and then just to the left of this point. Calculate the value of  $\frac{dS}{dx} = \Sigma CW$  for each of these successive positions of loading. If the sign of  $\frac{dS}{dx}$  changes from  $-$  to  $+$ , a position of loading for numerically maximum negative stress is determined.

It will be noted that the negative coefficients  $C$  occur at those salient points where the angles of the influence line point *upward*, while the positive coefficients  $C$  occur at those salient points where the angles point *downward*.

It is unnecessary to seek a position of loading for *maximum positive* stress by placing a wheel successively to the right and to the left of any salient point which has a *positive* coefficient; for if  $\frac{dS}{dx} = \Sigma CW$  be  $+$  when the wheel is to the right of this point, it would have a still larger  $+$

value when the wheel is to the left of the point. A change, therefore, of  $\frac{dS}{dx}$  from + to - would not result. Similarly, it may be shown to be unnecessary to seek a numerically *maximum negative* stress by trying wheels at any salient point which has a *negative* coefficient.

Formulas (7) and (8) are the general formulas for the solution of the sum of the ordinate-load products of an influence line and the rate of change of this sum, and are applicable to any form of influence line. They give at once a definite solution of the position of a set of loads producing maximum positive and negative stresses in any member of any truss or girder for which an influence line can be drawn and the values of such stresses. The method is particularly advantageous in the case of statically indeterminate structures, such as two-hinged and no-hinged arches, swing bridges, continuous girders, etc., where general analytical criteria for the positions of loads producing maximum stresses cannot readily be expressed and where such maximum stresses have had to be found by assuming positions of loadings and scaling the influence-line ordinates under all the loads, a laborious process and one open to much liability of mechanical inaccuracy.

In applying the present method to the simple forms of girders and trusses (viz., the statically determinate structures where the ordinates of the influence lines are readily expressible algebraically) it will generally be more convenient to transform formulas (7) and (8) in each case whereby the coefficients  $C$  may be expressed in terms of the geometric proportions of the truss or girder. This, in the following articles (4 to 7 inclusive), we shall proceed to do for the case of girder bridges (with and without panels), pier reactions, and through Pratt trusses with curved or horizontal chords. The general method will, however, be applied *directly* to the case of the three-hinged arch in Art. 8, which will serve as a typical example of the application of the method to any influence line.

## ARTICLE IV.

### GIRDER BRIDGE WITHOUT PANELS.

IN Fig. 4 is shown a girder bridge without panels. The live load has advanced beyond the span, this being the

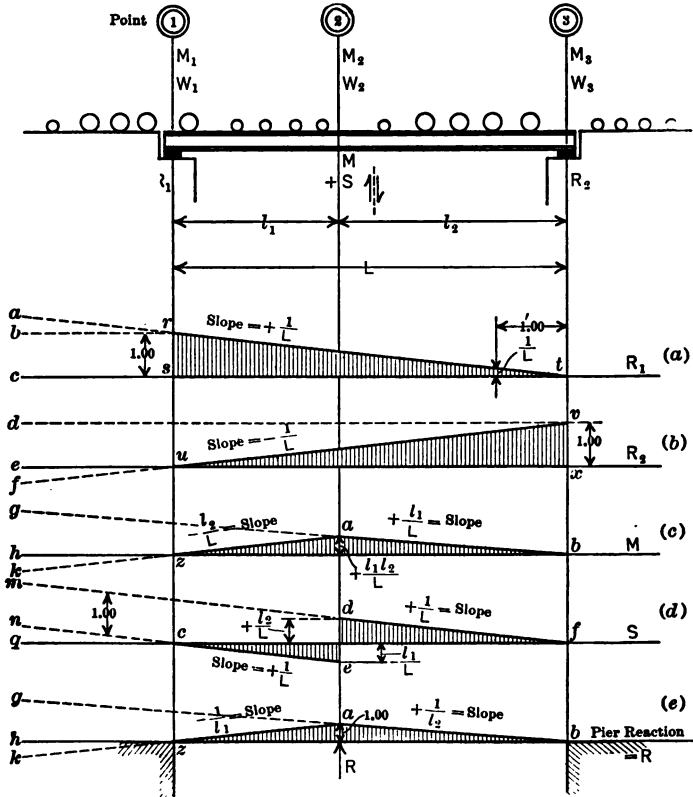


FIG. 4.

most general case. Formulas for the end reactions and for the bending moment and shear at any section will be developed.

The influence line for  $R_1$  is shown in Fig. 4a. The sum of the ordinate-load products within the shaded area  $rst$  equals the end reaction  $R_1$ , which at the same time is the end shear at  $R_1$ .

From Fig. 4a,

$$\begin{array}{cccccc} \text{Ordinate-load products in } & \underline{rst} = & & & & \\ & \text{"} & \text{"} & \text{"} & \text{"} & \underline{atc} \\ \text{---} & \text{"} & \text{"} & \text{"} & \text{"} & \underline{arb} \\ \text{---} & \text{"} & \text{"} & \text{"} & \text{"} & \underline{brsc}. \end{array}$$

By using formulas (4) and (5), this equation becomes

$$R_1 = \frac{1}{L} M_3 - \frac{1}{L} M_1 - W_1 = \frac{M_3 - M_1}{L} - W_1 \quad (9)$$

Any value of  $M$  or  $W$  may be read directly from Table 2 for the standard loadings given in Table 1. For example, in Fig. 4, if  $l_1 = 10'$ ,  $l_2 = 30'$ , and  $w_1$  of Cooper's  $E50$  has advanced 14' beyond the left end of the span, we have from Table 2,

$$\begin{array}{ll} \text{At 1, 14' from } w_1, M_1 = 350.0^K & W_1 = 62.50^K \\ \text{At 2, 24' from } w_1, M_2 = 1150.0 & W_2 = 112.50 \\ \text{At 3, 54' from } w_1, M_3 = 5435.0 & W_3 = 177.50 \end{array}$$

The formula for  $R_2$  is developed as for  $R_1$ , the method of writing the second member of the first equation being abbreviated in a way readily understood. From the influence line in Fig. 4b, and the formulas (4) and (5),

$$R_2 = \text{Ordinate-load products in } (\underline{dvxe} - \underline{dvf} + \underline{fue})$$

Or

$$R_2 = W_3 - \frac{1}{L} M_3 + \frac{1}{L} M_1 = W_3 - \frac{M_3 - M_1}{L} \quad (9a)$$

The sum of the reactions  $R_1$  and  $R_2$  as given by (9) and (9a) equals  $W_3 - W_1$ , or the sum of the loads on the bridge.

From the influence line in Fig. 4c and formulas (5) or (7), the equation for bending moment may be written:

$$M = \text{Ordinate-load products in } (\underline{gbh} - \underline{gak} + \underline{kzh})$$

or

$$M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \quad . . . . (10)$$

Formula (10) readily follows, likewise, from the general formula (7),  $S = C_1 M_1 + C_2 M_2 + C_3 M_3 = \Sigma CM$ .

For example, in the case of the bending moment at point 2 in Fig. 4,

$$\begin{aligned} C_1 &= 0 + \frac{l_2}{L} \\ C_2 &= -\frac{l_2}{L} - \frac{l_1}{L} = -1 \\ C_3 &= \frac{l_1}{L} - 0 \end{aligned}$$

Vhence 
$$M = \frac{l_2}{L} M_1 - M_2 + \frac{l_1}{L} M_3 \quad . . . . (10a)$$

Taking the derivative of  $M$  with respect to the advance  $x$  of the loading toward the left or using formula (8) directly, the rate of variation of the bending moment is

$$\frac{dM}{dx} = \frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2 \quad . . . . (11)$$

All positions for maximum  $M$  may be found by trying wheels at point 2 as directed by Rule 1 of Art. 3. In applying this rule the simultaneous shifting of other wheels of the rigid loading from right to left of points 1 and 3 as a wheel is shifted from right to left of point 2, must be taken into account by substituting in formula (11) the corresponding changed values of  $W_1$  and  $W_3$ . It is to be remembered, as stated in Art 3, that it is entirely unnecessary to try wheels at points 1 and 3.

From the influence line in Fig. 4d, the formula for the intermediate shear  $S$  follows by applying formulas (4) and (5):

$S = \text{Ordinate-load products in}$

$$(\underline{mfq} - mden - \underline{ncq})$$



Or

$$S = \frac{1}{L} M_3 - W_2 - \frac{1}{L} M_1 = \frac{M_3 - M_1}{L} - W_2 \quad (12)$$

There is one more thing to be borne in mind in calculating maximum bending moments in a girder bridge without panels: it is the rule for finding the section where the *absolute maximum bending moment* occurs. The rule is often spoken of as the "*centre of gravity rule*," and may be stated as follows:

*The bending moment under any given wheel becomes maximum when the centre of the span bisects the distance from the wheel in question to the centre of gravity of the loading on the span.*

In the practical application of this rule, the procedure is first to find the wheel which gives maximum bending moment at the centre of the span and then to shift this wheel so that the bending moment beneath it becomes an absolute maximum according to the centre of gravity rule. For the usual standard loadings the maximum centre moment closely approximates the absolute maximum bending moment for the spans greater than 70 feet.

The proof of the centre of gravity rule follows. Refer to Fig. 5. Assume that it has been found by trial that the wheel  $w_n$  gives the maximum centre moment. The general case where load has advanced beyond the span is taken. In order to get an *absolute maximum* bending moment under  $w_n$ , this wheel must be shifted a certain distance from the centre. Let such position be distance  $z$  from  $R_1$ . The sum of the loads on the span is called  $P$ , and equals  $(W_3 - W_1)$ . The centre of gravity of the load  $P_2$  is distance  $\bar{x}$  from  $R_2$ . The sum of the loads on the span to the left of  $w_n$  is called  $P_1$ , and their centre of gravity is at the fixed distance  $b$  from  $w_n$ .

Taking moments about  $R_2$ ,

$$R_1 = \frac{P_2 \bar{x}}{L}$$

Therefore,

$$M = R_1 y - P_1 b = \frac{P_2 \bar{x}}{L} y - P_1 b.$$

In this equation for  $M$ , the only variables are  $\bar{x}$  and  $y$ . Therefore,  $M$  will be a maximum when the product  $\bar{x}y$  is maximum. Note, however, that the sum

$$\bar{x} + y = (L - a) = \text{constant}.$$

If two variables have a constant sum, their product is maximum when the two variables are equal. Therefore,  $M$  is maximum when  $\bar{x} = y$ . But when  $\bar{x} = y$ , the distance from  $w_n$  to the centre of gravity of the loading is bisected

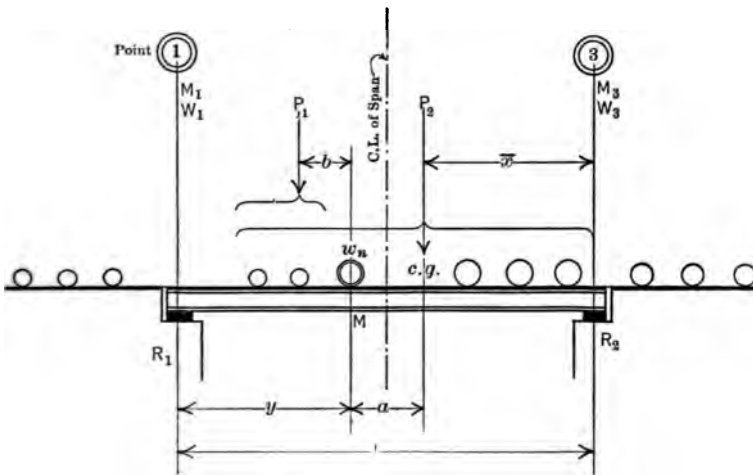


FIG. 5.

by the centre of the span. This proves the centre of gravity rule.

In order to apply this rule, a general expression for  $\bar{x}$  is needed.

Since  $R_1 = \frac{P_2 \bar{x}}{L}$  it follows that  $\bar{x} = \frac{R_1 L}{P_2}$ . Substitute the value of  $R_1$  from formula (9), and the value  $(W_3 - W_1)$  for  $P_2$ .

$$\bar{x} = \frac{M_3 - M_1 - L W_1}{W_3 - W_1} \quad \dots \dots (13)$$

In the special case where the loading has not advanced beyond the left end of the span,  $M_1$  and  $W_1$  equal zero and  $\bar{x}$  becomes

$$\bar{x} = \frac{M_3}{W_3} \quad \dots \dots \dots (13a)$$

Problems relating to a girder bridge without panels will now be given to illustrate the application of the above formulas and the use of some of the tables following the text.

*Problem.*—Given a 40-foot deck-girder bridge consisting of one girder per rail. Use Cooper's *E50* loading. Find the maximum shear at the end, quarter point, and centre. Determine also the maximum bending moment at the quarter point and at the centre, and the absolute maximum bending moment. All values are to be given *per rail*.

*Solution.*—Table 5 following the text gives the position of Cooper's loadings for maximum end shear. This table is the result of the solution of end shears for a large number of spans. As a general rule, however, it is safe to assume that  $w_2$  of Cooper's and similar loadings will always give the maximum end or intermediate shear when placed immediately to the right of the given section, the live load being headed toward the left. The exceptions in Table 5 to this general rule are not of prime importance, for the actual value of the shear when  $w_2$  is used is sufficiently close to the maximum even in the exceptional cases. There is no satisfactory criterion for determining the position of loading for maximum shear in girder bridges without panels, for it is as easy to calculate the actual values of the shears for the successive positions of loading as it is to apply any criterion. In the case of bending moment, however, time is saved by using the criterion.

*Maximum End Shear.*

Use formula (9),  $R_1 = \frac{M_3 - M_1}{L} - W_1$ . Place wheel 2

of Cooper's *E50* immediately to right of  $R_1$ . Take the values of moment and load sums for Cooper's *E50* from Table 2.

$$\text{Maximum end shear} = \frac{4370 - 100}{40} - 12.5 = 94.3^k.$$

*Maximum Shear at Quarter Point.*

Use formula (12) with  $w_2$  at quarter point.

$$S = \frac{M_3 - M_1}{L} - W_2$$

$$S \text{ at } \frac{1}{4} \text{ point} = \frac{2838.75 - 0}{40} - 12.5 = 58.5^k.$$

*Maximum Shear at Centre.*

Using formula (12) with  $w_2$  at centre.

$$S \text{ at centre} = \frac{1600 - 0}{40} - 12.5 = 27.5^k.$$

The values for the shears are given in Kips, or thousand of pounds. A comparison of the above shears with those in Table 7 shows agreement of results.

*Maximum Bending Moment at the One-Quarter Point.*

First compute successive pairs of values for  $\frac{dM}{dx}$  for different wheels, first placed to the right and then to the left of the quarter point. A change of sign from + to - indicates a wheel that gives a maximum. Use formula (11),

$$\frac{dM}{dx} = \frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2 \quad . . . . (11)$$

$w_1$  at  $\frac{1}{4}$  point.

$$\frac{dM}{dx} = \frac{1}{4} (112.5) + \frac{3}{4} (0) - 0 = +$$

No maximum.



with Table 11 shows agreement of results. Reference to Table 3 indicates that the correct wheel for maximum has been chosen.

*Maximum Bending Moment at the Centre.*

$$\frac{dM}{dx} = \frac{W_3 + W_1}{2} - W_2, (10a), \text{ and}$$

$$M = \frac{M_3 + M_1}{2} - M_2, (11a), \text{ when } \frac{l_1}{L} = \frac{1}{2}$$

$w_3$  at centre,

$$\frac{dM}{dx} = \frac{128.75}{2} - 37.5 = +$$

No maximum.

$$\frac{dM}{dx} = \frac{128.75}{2} - 62.5 = +$$

$w_4$  at centre,

$$\frac{dM}{dx} = \frac{145}{2} - 62.5 = +$$

Maximum.

$$\frac{dM}{dx} = \frac{145}{2} - 87.5 = -$$

$w_5$  at centre,

$$\frac{dM}{dx} = \frac{145 + 12.5}{2} - 87.5 = -$$

No maximum.

$$\frac{dM}{dx} = \frac{161.25 + 12.5}{2} - 112.5 = -$$

Therefore, maximum centre moment occurs with  $w_4$  at centre.

$$M = \frac{2838.75}{2} - 600 = 819.37 \text{ Kip feet.}$$

This value agrees with Table 11; and the position of loading, with Table 3.

*Absolute Maximum Bending Moment.*

Shift  $w_4$  according to centre of gravity rule, and then recompute the value of  $M$  under this wheel by formula (10). Note that new values for  $l_1$ ,  $l_2$ , and  $M_3$  must be determined.

By formula (13a), when  $w_4$  is at the centre,

$$\bar{x} = \frac{M_3}{W_3} = \frac{2838.75}{145} = 19'.58$$

Therefore for absolute maximum bending moment under

$$w_4, \text{ shift loading to left } \frac{20'.00 - 19'.58}{2} = 0'.21.$$

The new values of  $l_1$ ,  $l_2$ , and  $M_3$  are

$$l_1 = 20.00 - 0.21 = 19.79$$

$$l_2 = 20.00 + 0.21 = 20.21$$

$$M_3 = 2838.75 + .21(145) = 2869.2$$

The absolute maximum bending moment =

$$\begin{aligned} M &= \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \\ &= \frac{19.79}{40} (2869.2) + 0 - 600 = 819.54 \text{ Kip feet.} \end{aligned}$$

It appears, therefore, that the absolute maximum bending moment is .17 Kip feet greater than the maximum centre moment. The difference is not great in this particular case, as the required shift of the loading is comparatively small. The position of loading for absolute maximum bending moment agrees with Table 4, and its value agrees with Table 7.

## ARTICLE V.

### PIER REACTION.

IN Fig. 4e is given the influence line for the pier reaction  $R$  between two non-continuous beam spans  $l_1$  and  $l_2$ . From this influence line, the formulas (5) and (7) give

$$R = \text{Ordinate-load products in } (\underline{gbh} - \underline{gak} + \underline{kzh})$$

Or,

$$R = \frac{M_3}{l_2} + \frac{M_1}{l_1} - \frac{L}{l_1 l_2} M_2 = \frac{L}{l_1 l_2} \left( \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \right) \quad (14)$$

Formula (14) may also be derived from formula (10) since the ordinates of the influence line for  $R$  bear the constant ratio  $\frac{L}{l_1 l_2}$  to the corresponding influence ordinates for  $M$ , the position of the live load and the values of  $l_1$  and  $l_2$  remaining fixed.

Therefore,

$$R = \frac{L}{l_1 l_2} M \quad . . . . . (16)$$

Substituting the value  $M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2$  from formula (10) in formula (16), the result is again formula (14).

For equal spans,

$$l_1 = l_2 = l \text{ so that } R = \frac{M_3 + M_1 - 2M_2}{l} \quad . \quad (14a)$$

The rate of change of  $R$  for a movement  $dx$  of the loading to the left is

$$\frac{dR}{dx} = \frac{W_3}{l_1} + \frac{W_1}{l_1} - \frac{L}{l_1 l_2} W_2 = \frac{L}{l_1 l_2} \left( \frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2 \right) \quad (15)$$



For equal spans,  $l_1 = l_2 = l$ , so that

$$\frac{dR}{dx} = \frac{W_3 + W_1 - 2W_2}{l} \dots \dots \dots (15a)$$

In the last member of formula (15) the quantity within the parentheses is the same as the expression for  $\frac{dM}{dx}$  in formula (11). It follows, therefore, that the same position of loading gives maximum  $R$  and maximum  $M$  for any given values of  $l_1$  and  $l_2$ .

*Problem.*—(a) Find the maximum pier reaction per rail between two simple beam spans  $l_1 = 10$  ft. and  $l_2 = 30$  ft. (b) Find the maximum pier reaction between two simple beam spans, each having a length of 20 feet. Use Cooper's E50 loading.

*Solution of Problem (a).*

Use formula (15) to find position of loading for maximum  $R$ .

$$\frac{dR}{dx} = \frac{L}{l_1 l_2} \left( \frac{l}{L} W_3 + \frac{l_2}{L} W_1 - W_2 \right) \dots (15)$$

$w_2$  at pier.

$$\frac{dR}{dx} = \frac{40}{10 \times 30} \left( \frac{10}{40} (145) + \frac{30}{40} (0) - 12.5 \right) = +$$

Maximum.

$$\frac{dR}{dx} = \frac{40}{10 \times 30} \left( \frac{10}{40} (145) + \frac{30}{40} (0) - 37.5 \right) = -$$

$w_3$  at pier.

$$\frac{dR}{dx} = \frac{40}{10 \times 30} \left( \frac{10}{40} (145) + \frac{30}{40} (12.5) - 37.5 \right) = +$$

Maximum.

$$\frac{dR}{dx} = \frac{40}{10 \times 30} \left( \frac{10}{40} (161.25) + \frac{30}{40} (12.5) - 62.5 \right) = -$$

Use formula (14) to compute the value of  $R$ .

$$R = \frac{M_3}{l_2} + \frac{M_1}{l_1} - \frac{L}{l_1 l_2} M_2.$$

t pier.

$$R = \frac{2838.75}{30} + \frac{0}{10} - \frac{40}{10 \times 30} (100) = 81^k.$$

t pier.

$$R = \frac{3563.75}{30} + \frac{37.5}{10} - \frac{40}{10 \times 30} (287.5) = 84^k.$$

The latter value of  $84^k$  is the maximum pier reaction. value agrees with Table 14 and the position of loading es with Table 3.

*Solution of Problem (b).*

Use formulas (14a) and (15a),

$$R = \frac{M_3 + M_1 - 2M_2}{l}, \text{ and } \frac{dR}{dx} = \frac{W_3 + W_1 - 2W_2}{l}.$$

t pier.

$$\frac{dR}{dx} = \frac{128.75 + 0 - 2 \times 37.5}{20} = +$$

No maximum.

$$\frac{dR}{dx} = \frac{128.75 + 0 - 2 \times 62.5}{20} = +$$

t pier.

$$\frac{dR}{dx} = \frac{145 + 0 - 2 \times 62.5}{20} = +$$

Maximum.

$$\frac{dR}{dx} = \frac{145 + 0 - 2 \times 87.5}{20} = -$$

t pier.

$$\frac{dR}{dx} = \frac{145 + 12.5 - 2 \times 87.5}{20} = -$$

No maximum.

$$\frac{dR}{dx} = \frac{161.25 + 12.5 - 2 \times 112.5}{20} = -$$

efore, maximum pier reaction occurs when  $w_4$  is at the

$$R = \frac{2838.75 - 0 - 2 \times 600}{20} = 81.9^k.$$

This maximum pier reaction of 81.9<sup>k</sup> agrees with value in Table 7 and Table 14, while the position of loading agrees with that given by Table 3.

## ARTICLE VI.

### GIRDER BRIDGE WITH PANELS.

In Fig. 6 is shown a girder bridge with panels. It is as-

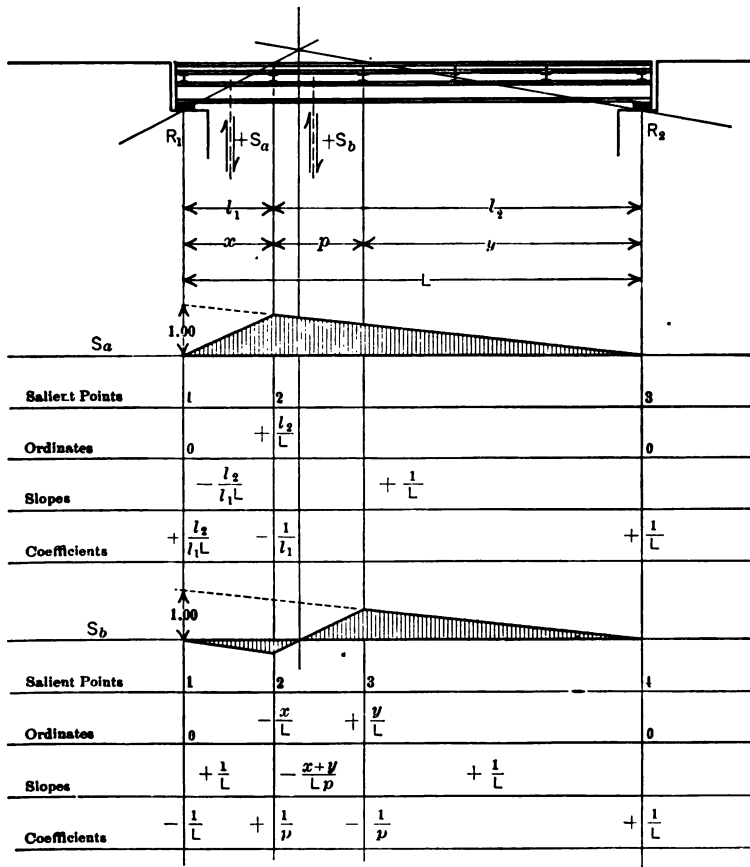


FIG. 6.

sumed that the live load has advanced beyond the left end of the span, this being the most general case.

The formulas for  $R_1$  and  $R_2$  are the same as formulas (9) and (9a) for the girder without panels, if the girder bridge with panels has end floor-beams; but if this bridge has end struts with the end stringers resting on separate pedestals, the value of  $R_1$  beneath the end of the main girder is the same as  $S_a$ , the shear in the end panel, as given by formula (17) to follow.

Inasmuch as the maximum bending moment in a beam carrying concentrated loads always occurs beneath a concentration, the maximum bending moments in the main girder of a girder bridge with panels will occur at the floor-beams. The influence line for the bending moment at the floor-beams is the same as for the bending moment in a girder bridge without panels; accordingly, formulas (10) and (11) are to be used in finding maximum bending moments at the floor-beams.

It remains to derive formulas for the maximum shears  $S_a$  in the end panel and  $S_b$  in any intermediate panel. In Fig. 6 are given the influence lines for  $S_a$  and  $S_b$ . The correctness of the ordinates is at once evident. The slopes and coefficients are calculated as explained in Arts. 2 and 3. The general formulas for  $S_a$  and  $S_b$  and their rates of variation may be written at once by use of formulas (7) and (8).

$$S_a = \frac{1}{L} M_3 + \frac{l_2}{l_1 L} M_1 - \frac{1}{l_1} M_2 = \frac{1}{l_1} \left( \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \right) \quad (17)$$

$$\frac{dS_a}{dx} = \frac{1}{L} W_3 + \frac{l_2}{l_1 L} W_1 - \frac{1}{l_1} W_2 = \frac{1}{l_1} \left( \frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2 \right) \quad (18)$$

$$S_b = \frac{1}{L} M_4 - \frac{1}{p} M_3 + \frac{1}{p} M_2 - \frac{1}{L} M_1 \quad \dots \dots \dots (19)$$

$$\frac{dS_b}{dx} = \frac{1}{L} W_4 - \frac{1}{p} W_3 + \frac{1}{p} W_2 - \frac{1}{L} W_1 \quad \dots \dots \dots (20)$$

Formula (17) when compared with formula (10) shows that  $S_a$  is equal to the bending moment at the first intermediate floor-beam divided by the length of the first panel. Formula (18) when compared with formula (11) shows that

the same position of loading that gives maximum bending moment at the first intermediate floor-beam will also give maximum shear in the end panel.

Formulas (19) and (20) are perfectly general and will serve for any assumed series of vertical loads in any position. For the usual standard loadings and panel lengths, however, it is not necessary to advance any loads beyond an intermediate panel for maximum shear in this panel. Therefore, for practical purposes formulas (19a) and (20a)

$$S_b = \frac{M_4}{L} - \frac{M_3}{p} = \frac{1}{p} \left( \frac{p}{L} M_4 - M_3 \right) \quad \dots (19a)$$

$$\frac{dS_b}{dx} = \frac{W_4}{L} - \frac{W_3}{p} = \frac{1}{p} \left( \frac{p}{L} W_4 - W_3 \right) \quad \dots (20a)$$

*Illustrative Problem.*—A single track through girder bridge with a floor system consisting of stringers and floor-beams, both end and intermediate, has six panels of 20 feet each. Find the maximum end reaction and the shear in panels 0 – 1, 1 – 2, and 2 – 3, using Cooper's E50 loading.

*Solution.*—For maximum end reaction place wheel 2 at left end. Use formula

$$R_1 = \frac{M_3 - M_1}{L} - W_1 \quad \dots \dots \dots (9)$$

$$R_1 = \frac{27651 - 100}{120} - 12.5 = 217.1^k$$

Note that the above value agrees with Table 7.

For maximum shear in panel 0 – 1, find critical wheel by formula (18) and then compute shear by formula (17).

Try wheel 3 at panel point 1.

$$\frac{dS_a}{dx} = \frac{1}{20} \left( \frac{1}{6} (365) + 0 - 37.5 \right) = + \quad \text{Maximum.}$$

$$\frac{dS_a}{dx} = \frac{1}{20} \left( \frac{1}{6} (365) - 0 - 62.5 \right) = -$$

Note that the position of loading agrees with Table 1. For this position of loading formula (17) gives

$$S_a = \frac{1}{20} \left( \frac{1}{6} (21895) + 0 - 287.5 \right) = 168.1^k.$$

For maximum shears in the intermediate panels, determine the position of loading by formula (20a) and the shear by formula (19a).

$$\frac{dS_b}{dx} = \frac{1}{p} \left( \frac{p}{L} W_4 - W_3 \right) \quad \dots \dots \dots (20a)$$

$$S_b = \frac{1}{p} \left( \frac{p}{L} M_4 - M_3 \right) \quad \dots \dots \dots (19a)$$

Panel 1-2. Try wheel 3 at panel point 2.

$$\frac{dS_b}{dx} = \frac{1}{20} \left( \frac{1}{6} (306.25) - 37.5 \right) = +$$

Maximum.

$$\frac{dS_b}{dx} = \frac{1}{20} \left( \frac{1}{6} (322.50) - 62.5 \right) = -$$

$$S_b = \frac{1}{20} \left( \frac{1}{6} (15051.25) - 287.5 \right) = 111.0^k.$$

Panel 2-3. Try wheel 3 at panel point 3.

$$\frac{dS_b}{dx} = \frac{1}{20} \left( \frac{1}{6} (240) - 37.5 \right) = +$$

Maximum.

$$\frac{dS_b}{dx} = \frac{1}{20} \left( \frac{1}{6} (240) - 62.5 \right) = -$$

$$S_b = \frac{1}{20} \left( \frac{1}{6} (9345) - 287.5 \right) = 63.5^k.$$

The above values for shears agree with the values given by Table 9. The wheel for maximum shear in panels of girder and truss bridges is given in Table 6.

## ARTICLE VII.

### THROUGH PRATT TRUSS. GENERAL FORMULAS FOR LIVE-LOAD STRESSES AND THEIR RATE OF VARIATION. ILLUSTRATIVE PROBLEMS.

THE general formulas  $S = \Sigma CM$  and  $\frac{dS}{dx} = \Sigma CW$  may be used to write the equations for the live-load stresses in any member of a framed structure as soon as its influence

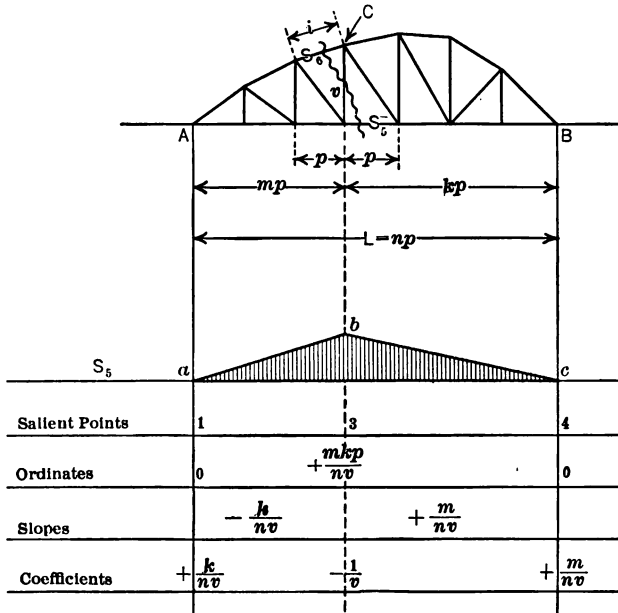


FIG. 7.

line has been drawn and the ordinates at the salient points determined.

In Figs. 7, 8, 9, and 10 are shown all the influence lines



needed in writing the formulas for the live-load stresses in a through Pratt truss with non-parallel or parallel chords. The influence ordinate at any salient point is the calculated stress due to a one-pound load on the bridge at the panel point above this salient point. By easily discovered relations between similar triangles, the algebraic value of each stress, or influence ordinate, is expressed in terms that are most readily evaluated in any numerical problem.

The derivation of any one formula for a live-load stress is typical. Refer to Fig. 7. The stress in the lower chord member  $S_5$  is found by taking moments about  $C$ . The influence line for  $S_5$  is straight over each of the two intervals  $kp$  and  $mp$ . The ordinates at the salient points 1 and 4 are zero. The ordinate at salient point 3 must be found by placing a one-pound load at the lower panel point of the truss above this salient point and calculating the value of  $S_5$ . For the unit load so placed,

$$\text{Reaction at } A = \frac{kp}{np} = \frac{k}{n}$$

By moments about  $C$ ,

$$\frac{k}{n}(mp) = S_5(v)$$

Therefore,

$$S_5 = + \frac{mkp}{nv} = \text{Influence ordinate at 3.}$$

The slopes of the segments of this influence line follow.

$$\text{Slope of } ab = - \frac{mkp}{nv} \div mp = - \frac{k}{nv}$$

$$\text{Slope of } bc = + \frac{mkp}{nv} \div kp = + \frac{m}{nv}$$

The coefficients  $C$  for use in the general formula  $S = \Sigma CM$  are now found.

$$C_1 = 0 + \frac{k}{nv} = + \frac{k}{nv}$$

$$C_3 = -\frac{k}{nv} - \frac{m}{nv} = -\frac{1}{v}$$

$$C_4 = \frac{m}{nv} - 0 = +\frac{m}{nv}$$

Therefore, for the position of the live load advanced beyond the limits of the span, the general formula for  $S_5$  is

$$S_5 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 + \left(\frac{k}{nv}\right)M_1.$$

However, in actual practice it is usually not necessary to advance the loading beyond the left end of the span in order to get a maximum value of  $S_5$ . The usual formula will therefore not contain the term  $M_1$ , since this will be zero; thus,

$$S_5 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 \quad \dots \dots (21)$$

Inasmuch as the horizontal component of the stress  $S_4$  in an inclined top chord member or end post equals the stress  $S_5$  in a corresponding lower chord member, the stress in any top chord member or end post may be found by

$$S_6 = \frac{i}{p} \cdot S_5 \quad \dots \dots (22)$$

In Fig. 8 is shown the influence line for the stress  $S_4$  in any vertical post. The influence ordinates are determined by taking moments about the intersection of the upper and lower chord members which are cut by the section. The algebraic values of these ordinates are transferred by use of easily discovered relations between similar triangles. The slopes and coefficients are then calculated in the usual way. The influence line indicates that the live load should advance into but not beyond the panel for a maximum compression, and for this reason  $M_1$  and  $M_2$  equal zero for the usual case. The numerical value of

the maximum *compression*  $S_4$  in a vertical post is, therefore,

$$S_4 = \left(\frac{a}{bL}\right)M_1 - \left(\frac{1}{p}\right)M_2 \quad \dots \quad (23)$$

The coefficients for the stress in any inclined web member are given by Fig. 9. The quantities for  $S_1$  and  $S_2$  are

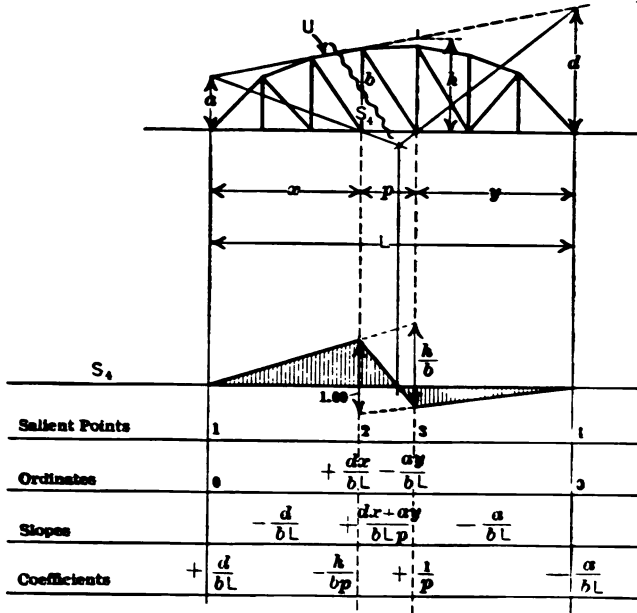


FIG. 8.

as shown, and the quantities for  $S_3$  are of the same algebraic form except that they are of opposite sign throughout. For the usual position of the live load advanced from the right into but not beyond the panel  $p$  for maximum stress, the moment sums  $M_1$  and  $M_2$  equal zero, and the numerical values of the maximum tension  $S_1$  and  $S_2$  and of the maximum compression  $S_3$  are given by the following formula:

$$S_1, S_2, \text{ or } S_3 = \left(\frac{ta}{cbL}\right)M_1 - \left(\frac{t}{bp}\right)M_2 \quad \dots \quad (24)$$

In a special case where the loading must be advanced beyond the panel  $p$  until the tension in the inclined counter-reb member  $S_2$  is balanced by the dead-load compression

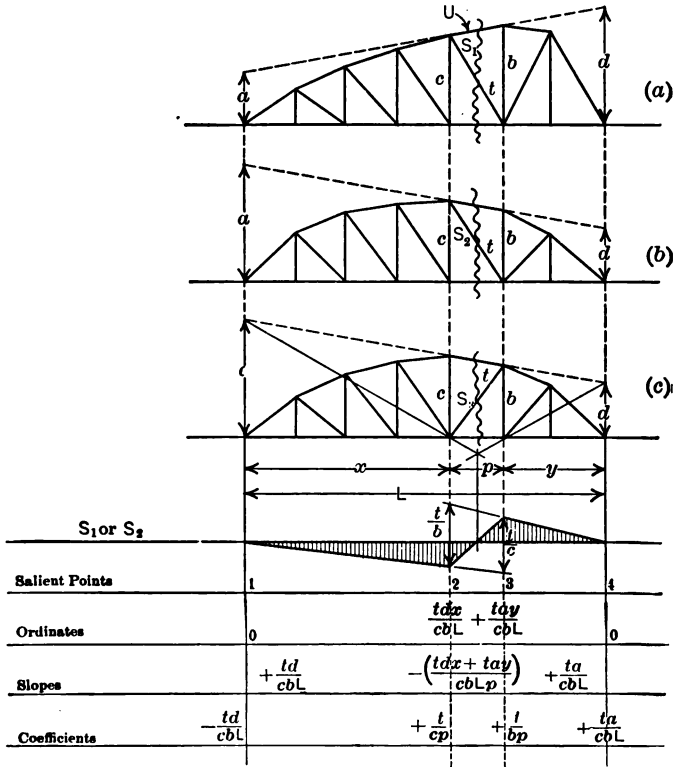


FIG. 9.

in this same member, the value of  $M_2$  is not zero, and the formula for  $S_2$  becomes

$$S_2 = \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)M_3 + \left(\frac{t}{cp}\right)M_2$$

$$\text{Or, letting } M_c = \left(M_3 - \frac{b}{c}M_2\right),$$

$$S_2 = \left(\frac{ta}{cbL}\right)M_4 - \frac{t}{bp}\left(M_3 - \frac{b}{c}M_2\right) = \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)M_c \quad (25)$$

Note that the coefficients of  $M_1$  and  $M_2$  in this formula are the same as the coefficients for  $M_1$  and  $M_3$  in formula (24).

The influence line for the counter-tension in a vertical post is shown in Fig. 10. For the usual case, the loading advances beyond the panel but not beyond the end of the span. Therefore  $M_1$  is equal to zero, so that

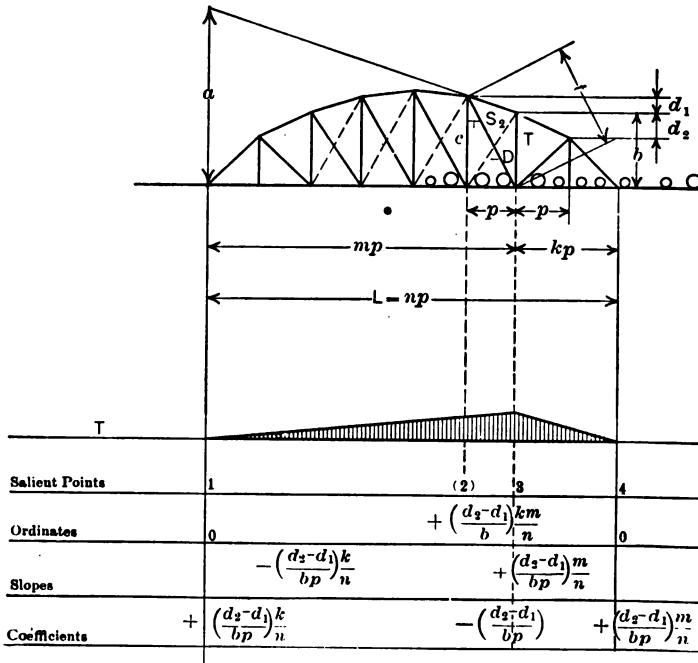


FIG. 10.

$$T = \left(\frac{d_2-d_1}{bp}\right)\left(\frac{m}{n}M_4 - M_3\right) = K \cdot M_o \quad . \quad . \quad (26)$$

where  $K$  and  $M_o$  stand for the corresponding terms in the parentheses. In order that  $T$  be a maximum the live load must advance beyond the position for the maximum tension  $S_2$  until the tension as computed by formula (25) becomes equal to the dead-load compression in this same member. For this position of the live load, the value of  $T$  is then computed by using formula (26). It may be noted that

some specifications state that only  $\frac{2}{3}$  of the dead-load compression is to be counted as effective in counteracting the live-load tension in an inclined counter-web member. This specification has been observed in the problem to follow.

A review of the preceding formulas shows that all the live-load stresses may be computed by formulas (21), (22), (23), and (24), except the counter-tension in a vertical post and the tension in a floor-beam hanger. Formula (25) makes it possible to find readily by trial the position of loading for maximum counter-tension in a vertical post, and formula (26) gives the value of this tension. The maximum tension in the floor-beam hanger may be found by the use of formulas (14a) and (15a) for pier reaction between equal spans.

If the chords of the Pratt truss are parallel, there will be no counter-tension in any vertical post. Formula (21) for the stress in a horizontal chord member and formula (22) for the stress in the inclined end post remain unchanged. Formulas (23) and (24) for web stresses are simplified because  $a = b = \text{depth of truss}$ .

The formulas, therefore, for the Pratt truss with parallel chords are:

Stress in horizontal chord members =

$$S_5 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 \dots \dots \dots (21)$$

$$\text{Stress in inclined end post} = S_6 = \frac{i}{p} S_5 \dots \dots \dots (22)$$

$$\text{Stress in vertical post} = S_4 = \left(\frac{1}{L}\right)M_4 - \left(\frac{1}{p}\right)M_3 \dots \dots (29)$$

Stress in inclined web member =

$$S_1 = \left(\frac{t}{cL}\right)M_4 - \left(\frac{t}{cp}\right)M_3 = \frac{t}{c} S_4 \dots \dots (30)$$

One general formula will suffice for finding the position of loading for maximum chord and web stresses of a Pratt truss with either inclined or parallel chords. The formulas

(21), (23), (24), (29), and (30) for these stresses are of general form

$$S = (G) M_1 - (H) M_2 \quad \dots \dots (2)$$

where  $G$  and  $H$  are the corresponding coefficients of  $M$

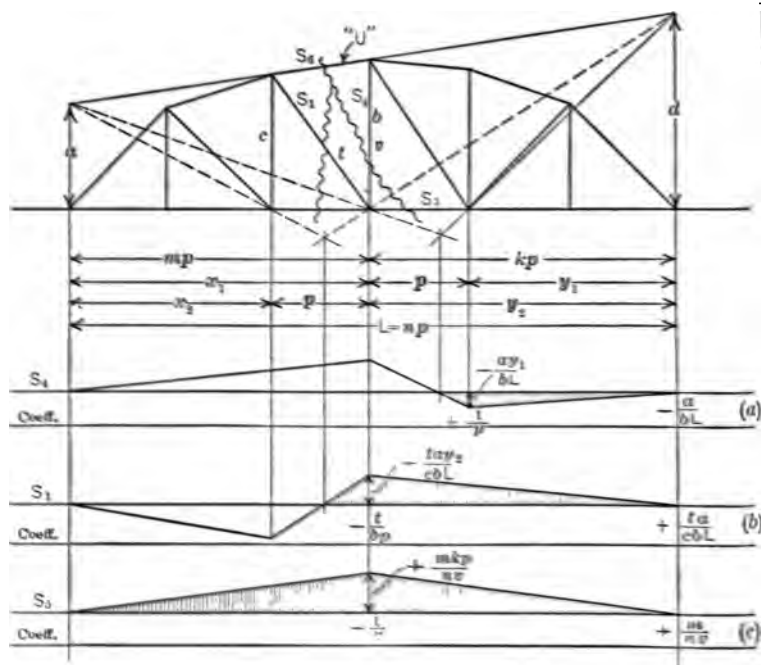


FIG. 11.

and  $M_2$  in the preceding formulas. The rate of variation of  $S$  as the load advances is

$$\frac{dS}{dx} = GW_1 - HW_2 = H \left( \frac{G}{H} W_1 - W_2 \right) \quad \dots (28)$$

When any one of the above stresses is a maximum, the value of  $\left( \frac{G}{H} W_1 - W_2 \right)$  passes through zero as a wheel is shifted from right to left of the salient point 3 in Figs. 7, 8, or 9.

The preceding formulas for the live-load stresses are summarized for convenient reference in Art. 11 preceding

the Tables. The important dimensions and quantities in Figs. 7, 8, and 9 are summarized in Fig. 11. If a uniform live load is used, the shaded areas in Fig. 11a, b and c multiplied by the intensity of the uniform load will give the maximum live-load stresses. The algebraic value of any one of these triangular areas is conveniently expressed as the base of the triangle times  $\frac{1}{2}$  of the given algebraic ordinate. The lengths of the bases of the shaded areas in Figs. 11a and b may be readily determined by one of the constructions shown in Figs. 12a and 12b, which give the position of the unit load for zero stress in the members indicated. The proofs that these constructions locate neutral points are not given, for they are generally known, and are proved in numerous texts on bridges. (See Marburg's "Framed Structures and Girders," Vol. I, page 392.)

The application of the preceding formulas will now be made to the calculation of the live-load stresses in the two single track through Pratt trusses shown in Figs. 13 and 14. A convenient procedure is as follows:

1. Determine the lengths of all inclined members and write their values on the truss outline.
2. Determine the values of the intercepts  $a$  as defined by Fig. 11 and write their values on the truss outline.
3. Write on the truss outline the distances of the several panel points from the right end of the span.
4. Write down the reciprocals of the span, panel length, and lengths of vertical members.
5. Make a form for tabulating calculations and list members in some convenient form as is done in Figs. 13 and 14.
6. Calculate the numerical values of the coefficients  $G$  and  $H$  for the several members by use of the formulas already derived.
7. Determine the position of the loading for maximum

stress by finding the position of loading causing  $\left(\frac{G}{H} W_4 - W_3\right)$



to pass through zero, and for this position of loading select from Table 2 the corresponding values of  $M_1$  and  $M_2$ . At

VARIOUS CONSTRUCTIONS USED TO FIND NEUTRAL POINTS IN PRATT TRUSSES.

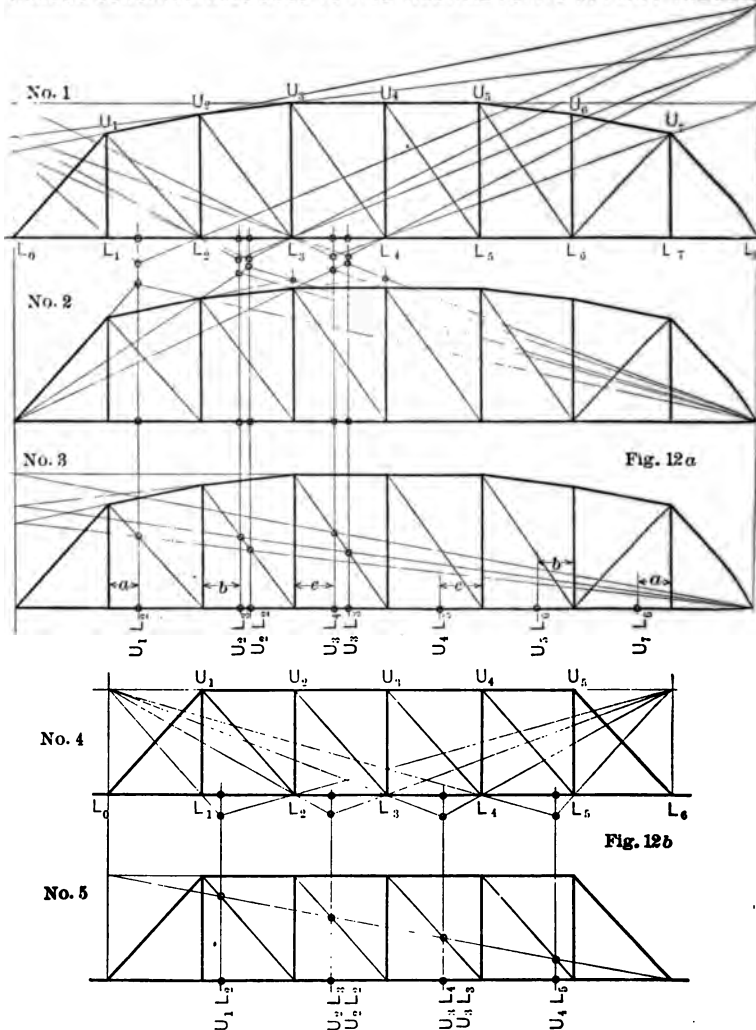


FIG. 12.

the same time tabulate the length  $L_1$  of loading causing maximum stress as this value is used in the impact formula

$$I = S \cdot \frac{300}{L_1 + 300}$$

8. Calculate values of  $S = GM_1 - HM_2$  and combine with impact and dead-load stresses. When the dead- and live-load stresses are of opposite sign, the combination is usually not algebraic but according to the particular specification that is used.

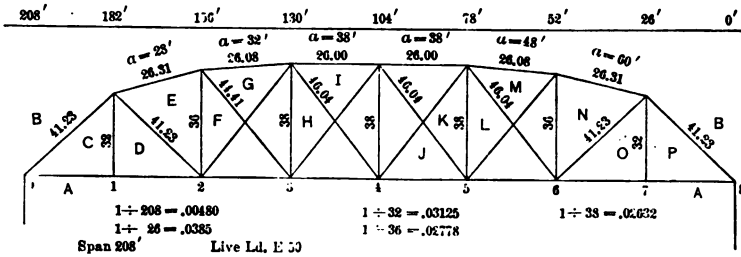


FIG. 13.

Mem.	G	H	Wheel	M <sub>1</sub>	M <sub>2</sub>	GM <sub>1</sub>	HM <sub>2</sub>	S	L <sub>1</sub>	$\frac{300}{L_1 + 300}$	I	DL	Total K
EF	.00373	.0385	3 @	333970	287	127	11	-116	143	.677	-78	-40	-234
ED	.00481	.0442	3 @	246255	287	223	13	+210	169	.640	+134	+83	+427
GH	.00405	.0385	2 @	421531	100	87	4	-83	112	.728	-60	-15	-158
GF	.00500	.0450	3 @	333970	287	170	13	+157	143	.677	+106	+48	+311
IJ	.00480	.0385	2 @	512940	100	62	4	-58	86	.777	-45	+7	.....
IH	.00580	.0466	3 @	423375	287	136	13	+123	117	.719	+88	+21	+232
JK	.00580	.0466	2 @	512940	100	75	5	+70	86	.777	+54	-21	.....
ML	.00777	.0493	2 @	66550	100	51	5	+46	60	.833	+38	-50	.....
NO	.01030	.0496	2 @	72307	100	24	5	-19	34	.898	-17	+83	No counter
AC=AD	.00390	.0312	4 @	163111	600	247	19	+228	200	.600	+137	+101	+466
BC	.....	.....	.....	.....	.....	.....	.....	-362	.....	.....	-217	-160	-739
AF	.00695	.0278	7 @	259095	2694	410	75	+335	193	.608	+203	+154	+692
BE	.....	.....	.....	.....	.....	.....	.....	-339	.....	.....	-206	-156	-701
AH	.00985	.0263	11 @	359661	7310	587	192	+395	194	.607	+239	+181	+815
BG	.....	.....	.....	.....	.....	.....	.....	-396	.....	.....	+240	-181	-817
BI	.01315	.0263	13 @	450901	9585	670	252	-418	178	.627	-262	-194	-874
CD	.0385	.0770	4 @	13725	600	144	46	+98	44	.872	+86	+25	+209

Post at	Mem.	M <sub>1</sub>	M <sub>2</sub>	S	I D	K	M <sub>1</sub>	T	L <sub>1</sub>	$\frac{300}{L_1 + 300}$	I	D.L.	Total
5	JK	22261	2390	+16	-14	.00203	11340	+23	114	.725	+17	+3	+43
6	ML	8865	687	+35	-34	.00214	5960	+13	71	.8	+10	+1	+24

9. Find positions of loading for maximum counter-tensions in posts and compute values by use of formulas (25) and (26).

### PROBLEM 1.

#### *Calculation of Live-load Stresses in a Pratt Truss with Inclined Chord.*

The complete data for this problem are given in Fig. 13. Items 1 to 5 of the above method of procedure need no explanation. The values of the coefficients  $G$  and  $H$ , the position of the loading for maximum stress, and the value of the maximum stress will be determined for several typical members; for example, vertical post, inclined web members, horizontal chords, end post, and inclined chords

#### *Vertical Post EF.*

$$\text{Formula} \quad S_4 = \left(\frac{a}{bL}\right) M_4 - \left(\frac{1}{p}\right) M_3 \dots \dots \dots (23)$$

Refer to Fig. 11 for definition of dimensions.

$$G = \frac{a}{bL} = \frac{28}{36} (.00480) = .00373$$

$$H = \frac{1}{p} = .0385$$

Try  $w_3$  at panel point 3. Use Table 2.  $L_1 = 143'$ .

$$\left(\frac{G}{H} W_4 - W_3\right) = \frac{.00373}{.03850} (440.0) - \frac{37.5}{62.5} = \frac{+}{-}$$

Therefore  $w_3$  at 3 gives a maximum.

$$S = GM_4 - HM_3 = .00373(33970) - .0385(287.5) \\ = 126.7 - 11.0 = 115.7^k$$

$$\text{Impact factor} = \frac{300}{L_1 + 300} = \frac{300}{443} = .677$$

$$\text{Impact stress} = .677 \times 115.7 = 78.3^k.$$

*Inclined Web Member ED.*

Formula 
$$S_1 = \left( \frac{ta}{cbL} \right) M_4 - \left( \frac{t}{bp} \right) M_3 \quad . . . . (24)$$

Refer to Fig. 11 for definition of dimensions.

$$G = \frac{ta}{cbL} = \frac{41.23 \times 28}{32 \times 36} (.00480) = .00481$$

$$H = \frac{t}{bp} = \frac{41.23}{36} (.0385) = .0442$$

Try  $w_3$  at panel point 2. Use Table 2.  $L_1 = 169'$ .

$$\left( \frac{G}{H} W_4 - W_3 \right) = \frac{.00481}{.0442} (505.0) - \frac{37.5}{62.5} = \begin{matrix} + \\ \text{or} \\ - \end{matrix}$$

Therefore  $w_3$  at 2 gives a maximum.

$$S = GM_4 - HM_3 = .00481(46255) - .0442(287.5) \\ = 223 - 13 = 210^k.$$

$$\text{Impact factor} = \frac{300}{469} = .640$$

$$\text{Impact stress} = .640 \times 210 = 134^k.$$

*Inclined Web Member ML.*

Formula 
$$S_2 = \left( \frac{ta}{cbL} \right) M_4 - \left( \frac{t}{bp} \right) M_3 \quad . . . . (24)$$

Refer to Fig. 9 or Fig. 11 for definition of dimensions.

$$G = \frac{ta}{cbL} = \frac{46.04 \times 48}{38 \times 36} (.00480) = .00777$$

$$H = \frac{t}{bp} = \frac{46.04}{36} (.0385) = .0493$$

Try  $w_2$  at panel point 6. Use Table 2.  $L_1 = 60'$ .

$$\left( \frac{G}{H} W_4 - W_3 \right) = \frac{.00777}{.0493} (190) - \frac{12.5}{37.5} = \begin{matrix} + \\ \text{or} \\ - \end{matrix}$$

Therefore  $w_2$  at 6 gives a maximum.

$$S = GM_4 - HM_3 = .00777(6550) - .0493(100) \\ = 51 - 5 = 46^k.$$

$$\text{Impact factor} = \frac{300}{360} = .833$$

$$\text{Impact stress} = .833 \times 46 = 38^k.$$

*Lower Chord Member AC = AD.*

$$\text{Formula} \quad S_5 = \left(\frac{m}{nv}\right) M_4 - \left(\frac{1}{v}\right) M_3 \quad \dots \quad (21)$$

Refer to Fig. 11 for definition of dimensions.

$$G = \frac{m}{nv} = \frac{1}{8} (.03125) = .00390$$

$$H = \frac{1}{v} = .0312$$

Try  $w_4$  at panel point 1. Use Table 2.  $L_1 = 200'$ .

$$\left(\frac{G}{H} W_4 - W_3\right) = \frac{.00390}{.0312} (582.5) - \frac{62.5}{87.5} = \frac{+}{-}$$

Therefore  $w_4$  at 1 gives a maximum.

$$S = GM_4 - HM_3 = .00390(63111) - .0312(600) \\ = 247 - 19 = 228^k.$$

$$\text{Impact factor} = \frac{300}{500} = .600$$

$$\text{Impact stress} = .600 \times 228 = 137^k.$$

*End of Post BC.*

$$\text{Formula} \quad S_6 = \frac{i}{p} S_5 \quad \dots \quad (22)$$

$$S_6 = \frac{41.23}{26} (228) = 362^k, \text{ and impact} = \frac{41.23}{26} (137) = 217^k.$$

*Lower Chord Member AH.*

$$\text{Formula} \quad S_5 = \left(\frac{m}{nv}\right) M_4 - \left(\frac{1}{v}\right) M_3 \quad \dots \quad (21)$$

Refer to Fig. 11 for definition of dimensions.

$$G = \frac{m}{nv} = \frac{3}{8} (.02632) = .00985$$

$$H = \frac{1}{v} = .0263$$

Try  $w_{11}$  at panel point 3. Use Table 2.  $L_1 = 194'$ .

$$\left( \frac{G}{H} W_4 - W_3 \right) = \frac{.00985}{.0263} (567.5) - \frac{190}{215} \begin{matrix} + \\ \text{or} \\ - \end{matrix}$$

Therefore  $w_{11}$  at 3 gives a maximum.

$$S = GM_4 - HM_3 = .00985(59661) - .0263(7310) \\ = 587 - 192 = 395^k.$$

$$\text{Impact stress} = \frac{300}{494} S = .607 \times 395 = 239^k.$$

*Top Chord Member BG.*

Formula  $S_6 = \frac{i}{p} S_5 \dots \dots \dots (22)$

$$S_6 = \frac{26.08}{26} (395) = 396^k.$$

$$\text{Impact} = \frac{26.08}{26} (239) = 240^k.$$

*Counter-Tension in Post at Panel Point 5.*

Formulas

$$S_2 = \text{Stress } JK = \left( \frac{ta}{cbL} \right) M_4 - \left( \frac{t}{bp} \right) \left( M_3 - \frac{b}{c} M_2 \right) \\ = \left( \frac{ta}{cbL} \right) M_4 - \left( \frac{t}{bp} \right) M_c \dots \dots \dots (25)$$

$T$  = tension in post.

$$= \left( \frac{d_2 - d_1}{bp} \right) \left( \frac{m}{n} M_4 - M_3 \right) = K \cdot M_o \quad (26)$$

Refer to Fig. 10 for definition of dimensions.

The calculation of the dead-load compression in  $JK$  is

not given, but the value is  $21^k$ . Two-thirds of this compression, or  $14^k$ , will be considered effective in counterbalancing the live-load tension in  $JK$ . The live load must be advanced beyond the position of maximum live-load tension in  $JK$  (i.e.,  $w_2$  at panel point 5) until  $S_2$ , or the stress in  $JK$ , equals  $14^k$ . This must be done by trial,  $S_2$  being figured each time by formula (25). It is found that when 114' of loading has advanced upon the bridge, this condition is approximately satisfied. For this position of loading

$$M_4 = 22261$$

$$M_e = \left( M_3 - \frac{b}{c} M_2 \right) = (2565 - 175) = 2390$$

$$G = \left( \frac{ta}{cbL} \right) = \frac{46.04 \times 38}{38 \times 38} (.00480) = .00580$$

$$H = \left( \frac{t}{bp} \right) = \frac{46.04}{38} (.0385) = .0466$$

Therefore,

$$S_2 = .00580(22261) - .0466(2390) = 16^k.$$

This value of  $S_2 = 16^k$  balances  $\frac{2}{3} D = -14^k$ , nearly enough for practical purposes. Therefore, compute  $T$  for this position of the live load.

$$T = \left( \frac{d_2 - d_1}{bp} \right) \left( \frac{m}{n} M_4 - M_3 \right) = K \cdot M_o$$

$$K = \frac{2 - 0}{38 \times 26} = .00203$$

$$M_o = \frac{5}{8} (22261) - 2565 = 11340$$

$$T = .00203(11340) = 23^k$$

$$\text{Impact factor} = \frac{300}{414} = .725$$

$$\text{Impact stress for } T = .725 \times 23 = 17^k.$$

## PROBLEM 2.

### *Live-load Stresses in a Pratt Truss with Parallel Chords.*

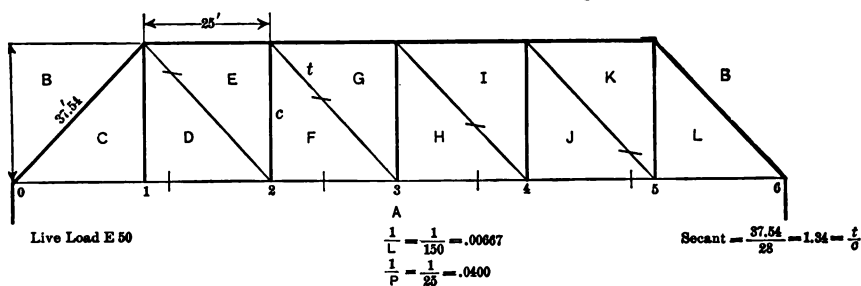
The complete data for this problem are given in Fig. 14. Formulas (21), (29), and (30) give the values of the

coefficients  $G$  and  $H$ , which are identical for several members of any Pratt truss with parallel chords. The procedure for finding the positions of the loading and maximum stresses is exactly as in Problem 1. It should be noted that

$$\text{Stress } FG = \text{Stress } EF \times \frac{37.54}{28}$$

$$HI = GH \times \frac{37.54}{28}$$

$$BC = AC \times \frac{37.54}{25}$$



**Fig. 14.**

Mem.	G	H	Wheel	M <sub>1</sub>	M <sub>2</sub>	S
CD	.0400	.0800	4 @ 1	3564	600	95
EF	.00667	.0400	3 " 3	13520	287	79
FG	.....	.....	.....	.....	.....	106
GH	.00667	.0400	2 " 4	6170	100	37
HI	.....	.....	.....	.....	.....	50
JK	.00894	.0536	2 " 5	2179	100	14
DE	.00894	.0536	3 " 2	21895	287	181
BC	.....	.....	.....	.....	.....	272
AC = AD	.00595	.0357	4 " 1	33970	600	181
AF = BE	.01190	.0357	7 " 2	31375	2694	278
BG	.01785	.0357	12 " 3	34411	8385	314

The stresses in all of the chord members may be checked by use of Table 8, and the stresses in the end post and web members may be checked by Table 9. The stress in  $CD$  agrees with the maximum pier reaction in Table 7. Table 3 may be used to find the position of loading for maximum chord stresses, and Table 6 gives position of loading for maximum web stresses.



## ARTICLE VIII.

### HIPPY-HINGED ARCH. APPLICATION OF THE GENERAL METHOD TO THE CALCULATION OF LIVE-LOAD STRESSES.

The general formulae  $\frac{\partial S}{\partial x} = \Sigma W$  and  $S = \Sigma CM$  may be used directly to find the position of loading and the

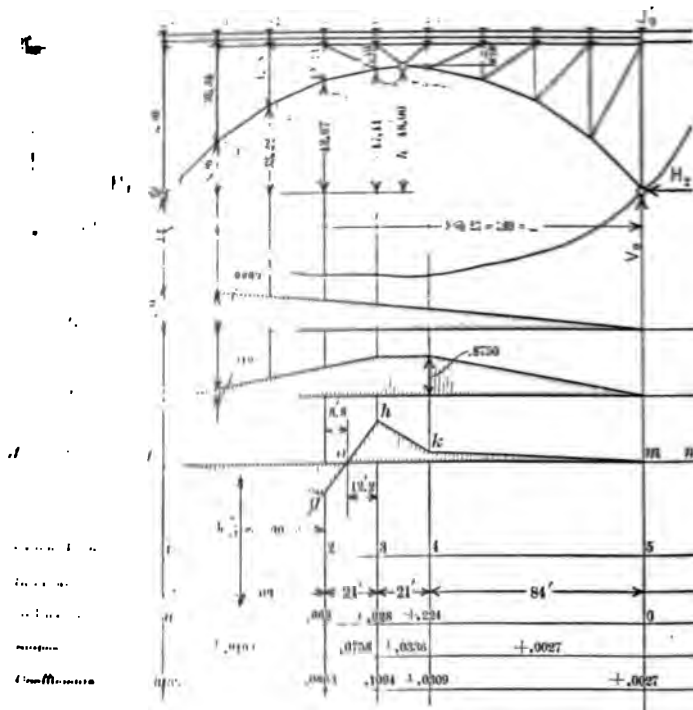


FIG. 15.

value of the maximum live-load stress in any member of a framed structure as soon as the influence line for this member and the ordinates at all salient points have been

determined. This method is applied to the calculation of maximum live-load stresses for the three-hinged arch shown in Fig. 15. Cooper's *E40* loading is used.

First are drawn the influence lines for the horizontal and vertical components of the reaction at the left hinge. The vertical component  $V_1$  is the same as for a simple span  $L$ . The horizontal component  $H_1$  equals the bending moment at the centre of the span  $L$  divided by the depth  $h$ . The influence-line ordinates for all members are now found by drawing five Maxwell diagrams, one of which is reproduced in Fig. 16. From the influence lines for  $V_1$  and  $H_1$ , the value of  $V_1$  is .8889 and  $H_1$  is .2187 for a one-pound load at  $U_1$ . The external loads acting on the left half of the arch are then as shown in Fig. 16a. The load line  $axbcya$  in Fig. 16b is drawn to a scale of  $10'' = 1$  pound, and the Maxwell diagram completed in the usual way. The scaled

TABLE A  
INFLUENCE-LINE ORDINATES FOR THREE-HINGED ARCH

Members	ORDINATES				
	1 lb. at $U_1$	1 lb. at $U_2$	1 lb. at $U_3$	1 lb. at $U_4$	1 lb. at $U'_4$
$U_0U_1 = \dots\dots\dots$	— .403	— .223	— .045	+ .130	+ .201
$U_1U_2 = \dots\dots\dots$	— .417	— .833	— .286	+ .262	+ .477
$U_2U_3 = \dots\dots\dots$	— .378	— .756	— 1.135	+ .189	+ .757
$U_3U_4 = \dots\dots\dots$	— .171	— .342	— .513	— .685	+ .548
$L_0L_1 = \dots\dots\dots$	— .295	— .590	— .885	— 1.180	— 1.182
$L_1L_2 = \dots\dots\dots$	+ .221	— .264	— .740	— 1.224	— 1.302
$L_2L_3 = \dots\dots\dots$	+ .217	+ .434	— .408	— 1.248	— 1.484
$L_3L_4 = \dots\dots\dots$	+ .164	+ .328	+ .491	— 1.086	— 1.674
$L_4L_5 = \dots\dots\dots$	— .048	— .096	— .145	— .193	— 1.420
$U_0L_0 = \dots\dots\dots$	— .692	— .384	— .075	+ .234	+ .345
$U_1L_1 = \dots\dots\dots$	— 1.014	— .632	— .253	+ .129	+ .287
$U_2L_2 = \dots\dots\dots$	+ .022	— .955	— .490	— .043	+ .165
$U_3L_3 = \dots\dots\dots$	+ .075	+ .150	— .775	— .317	— .076
$U_4L_4 = \dots\dots\dots$	+ .114	+ .226	+ .342	— .545	— .364
$U_0L_1 = \dots\dots\dots$	+ .800	+ .441	+ .085	— .270	— .400
$U_1L_2 = \dots\dots\dots$	+ .019	+ .878	+ .350	— .180	— .398
$U_2L_3 = \dots\dots\dots$	— .044	— .088	+ .986	+ .086	— .324
$U_3L_4 = \dots\dots\dots$	— .221	— .442	— .662	+ .928	+ .224
$U_4L_5 = \dots\dots\dots$	— .206	— .412	— .617	— .823	+ .657
$H \dots\dots\dots$	0.2187	0.4375	0.6562	0.8750	0.8750
$V \dots\dots\dots$	0.8889	0.7777	0.6666	0.5555	0.4444
$\theta \dots\dots\dots$	$14^\circ$	$29^\circ$	$44^\circ$	$58^\circ$	$63^\circ$

values of these stresses are the influence ordinates for a one pound load at  $U_1$ . In an exactly similar way the influence ordinates for a unit load at  $U_2$ ,  $U_3$ ,  $U_4$ , and  $U'_4$  are determined. The influence lines are straight from  $U'_0$  to

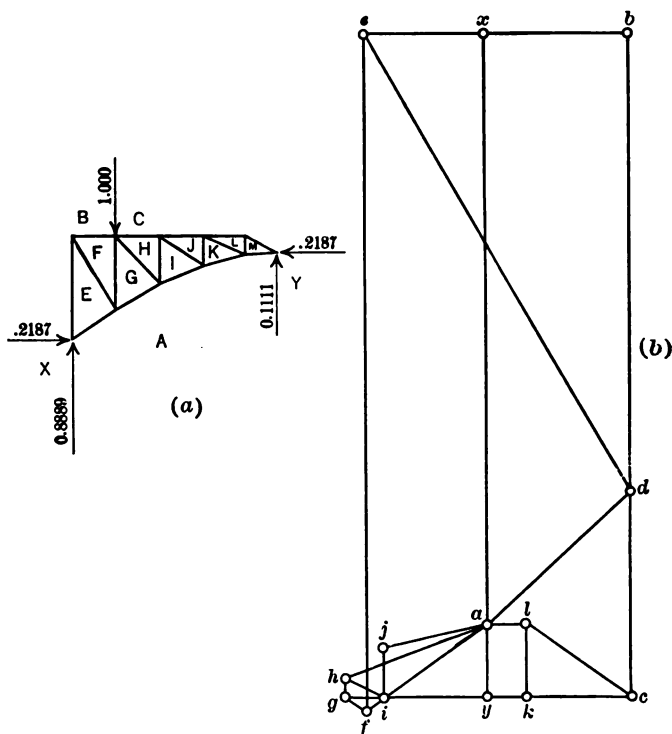


FIG 16.

$U'_4$ . Table A gives the influence ordinates for all members and also for the horizontal and vertical components of the reaction at the left hinge. The angle  $\theta$  is the inclination of this reaction with the vertical.

The calculation of the live-load stresses in any one member is typical. The member  $U_3L_4$  is taken. The influence line for this member is drawn to scale in Fig. 15 by use of the influence ordinates from Table A. The salient points occur below panel points  $U_3$ ,  $U_4$ , and  $U'_4$ . The distance

from  $U_3$  to the neutral point 0 equals  $\frac{.662}{.662 + .928} (21) = 8'.8$ .

*Calculation of Slopes.*

Slope of  $df = 0$

$$fg = \frac{0 - (-.662)}{68} = +.0105$$

$$gh = \frac{-.662 - (.928)}{21} = -.0758$$

$$hk = \frac{.928 - (.224)}{21} = +.0336$$

$$km = \frac{.224 - 0}{84} = +.0027$$

$$mn = 0$$

*Calculation of Coefficients.*

$$C_1 = 0 - (.0105) = -.0105$$

$$C_2 = .0105 - (-.0758) = +.0863$$

$$C_3 = -.0758 - (.0336) = -.1094$$

$$C_4 = .0336 - (.0027) = +.0309$$

$$C_5 = .0027 - 0 = +.0027$$

The sum of these coefficients equals zero. This agrees with formula (6) of Art. 3.

It should be remembered, as is pointed out in Art. 3, that the value of these coefficients may be measured graphically. For example, in Fig. 15 the value of  $C_2$  is  $\frac{2.59}{30} = .0863$ .

By use of the formula  $\frac{dS}{dx} = \Sigma CW$  and Rule 1 of Art.

3, the position of loading for maximum tension in  $U_3L_4$  may now be determined. Try wheel 3 at  $U_4$  with the loading advancing toward the left. Take the values of the load sums and moment sums for  $E40$  from Table 2.

$$\frac{dS}{dx} = \Sigma CW = -.1094(30) + .309(103) + .0027(302) = +.7$$

$$\frac{dS}{dx} = \Sigma CW = -.1094(50) + .309(103) + .0027(302) = -.7$$

Therefore  $w_3$  at  $U_4$  gives a maximum tension in  $U_3L_4$ , and its value is

$$S = \Sigma CM = -.1094(230) + .309(1846) + .0027(19001) = 83^k.$$

By use of the formula  $\frac{dS}{dx} = \Sigma CW$  and Rule 2 of Art. 3,

the position of loading for maximum compression in  $U_3L_4$  is now determined. Try wheel 2 at  $U_3$  with the loading advancing toward the right. Note that the signs of the coefficients remain unchanged. Take the values of the load sums and moment sums for  $E40$  from Table 2.

$$\frac{dS}{dx} = \Sigma CW = -.0105(192) + .0863(10) = -1.3$$

$$\frac{dS}{dx} = \Sigma CW = -.0105(192) + .0863(30) = +0.6$$

Therefore  $w_2$  at  $U_3$  gives a maximum negative stress, or compression, in  $U_3L_4$ , and its value is

$$S = \Sigma CM = -.0105(7092) + .0863(80) = -67^k.$$

The above values of  $83^k$  and  $67^k$  for maximum tension and compression in  $U_3L_4$  may be checked by use of formula  $S = qA_z$  (2), the values of  $q$  being taken from Table 16.

*Tension  $U_3L_4$  by Equivalent Uniform Load.*

The area of the tension part of the influence line equals

$$A_z = 27.2$$

The influence line  $ohkm$  is not triangular, but a triangular influence line with intervals  $l_1 = 10$  ft. and  $l_2 = 45$  ft. approximates its shape closely enough for the selection of an equivalent uniform load. For  $l_1 = 10'$  and  $l_2 = 45'$ , Table 16 gives  $3.080^k$  as the equivalent uniform load.

Therefore,

$$S = qA_s = (3.080) (27.2) = 84^k.$$

This value checks very closely that obtained by the exact method.

*Compression  $U_3L_4$  by Equivalent Uniform Load.*

Choose from Table 16 the equivalent uniform load for  $l_1 = 10$  ft. and  $l_2 = 65$  ft. From the influence line  $A_s = 23.7$ .

Therefore,

$$S = qA_s = (2.870) (23.7) = 68^k.$$

This checks closely the value obtained by the exact method.

Calculation of other members of this arch and of some more complicated framed structures shows a close agreement between the two preceding methods. The latter method is the simpler when a table of equivalent uniform loads has been made, especially in the case of the more complex influence lines for members of swing bridges, two-hinged arches, arch ribs, etc. The method of calculating a table of equivalent uniform loads will be explained in the following article.

## ARTICLE IX.

### EQUIVALENT UNIFORM LOADS.

AN equivalent uniform load is one which gives the same stress as does a loading which is not uniform. For any given standard loading, the equivalent uniform load is different for stresses whose influence lines differ. Since the forms of influence lines are innumerable, a table of exact equivalent uniform loads for all stresses is impracticable. A table of equivalent uniform loads, however, for stresses whose influence lines are *triangular* may be used with little error in selecting equivalent uniform loads for stresses whose influence lines are *not triangular*. It is, therefore, sufficient for practical purposes to make tables of equivalent uniform loads for a series of *triangular* influence lines. It may be shown that the equivalent uniform load for any triangular influence line is dependent entirely upon the intervals  $l_1$  and  $l_2$ , and is independent of the ordinate  $h$  at the apex of the influence line. Consider the triangular influence line in Fig. 1b to be for any stress  $S$ . Let the ordinate below  $C$  be any value  $h$ . If  $q$  equals the equivalent uniform load covering  $l_1$  and  $l_2$ ,

$$S = qA_z, \text{ or } q = \frac{S}{A_z} \quad . . . . . (A)$$

The area of this influence line is

$$A_z = \frac{h}{2} (l_1 + l_2) = \frac{h}{2} L \quad . . . . . (B)$$

Furthermore, if the concentrated live loads have been placed so as to give the maximum pier reaction between two spans  $l_1$  and  $l_2$ , this same position of loading will give maximum  $S$ , if the influence line for  $S$  is a triangle with the

same intervals  $l_1$  and  $l_2$ . Since the influence ordinates for  $S$  are related to the influence ordinates for  $R$  as  $h$  is to unity,

$$\frac{S}{R} = \frac{h}{1.00}$$

Or

$$S = hR \quad . . . . . (C)$$

Substituting the values of  $A_z$  and  $S$  from equations (B) and (C) in equation (A),

$$q = hR \div \frac{h}{2} L = \frac{2R}{L} \quad . . . . . (D)$$

It appears, therefore, that  $q$  is independent of  $h$ .

From formula (16) of Art. 5,

$$R = \frac{L}{l_1 l_2} M \quad . . . . . (16)$$

Substituting for  $R$  in equation (D),

$$q = \frac{2R}{L} = \frac{2M}{l_1 l_2} \quad . . . . . (31)$$

The term  $M$  is the bending moment in the span  $L = l_1 + l_2$  at the point where the intervals are  $l_1$  and  $l_2$ .

Tables (10) to (18) inclusive have been calculated for the positions of the live load given by Table 3. The values of  $M$  were first found, then the values of  $R$ , and finally the values of the equivalent uniform loads. The three formulas that were used in succession are

$$M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \quad . . . . . (10)$$

$$R = \frac{L}{l_1 l_2} M \quad . . . . . (16)$$

$$q = \frac{2M}{l_1 l_2} = \frac{2R}{L} \quad . . . . . (31)$$



An example of the use of equivalent uniform loads has already been given in Art. 8. The general formula  $S = qA$ , may be used in any case. For the special cases of bending moment in a beam and pier reaction between two simple spans, formula (31) gives

$$M = q \left( \frac{l_1 l_2}{2} \right) \dots \dots \dots (32)$$

$$R = q \left( \frac{L}{2} \right) = q \left( \frac{l_1 + l_2}{2} \right) \dots \dots \dots (33)$$

The quantities in the parentheses are the areas of the influence lines for  $M$  and  $R$  respectively.

## ARTICLE X.

### METHOD OF CALCULATING TABLE OF LOAD SUMS FOR ANY STANDARD LOADING. ILLUSTRATIVE EXAMPLE.

THE definitions of *moment sum* and *load sum* are given at the beginning of Art. 2. It is at once evident that a table of *load sums* may be computed by adding the successive loads. It may be shown that the table of *moment sums* may also be calculated by the process of addition.

From formula (5a) of Art. 2,

$$C_a W_a = C_a \frac{dM_a}{dx}$$

Or

$$dM_a = W_a \cdot dx.$$

Expressed in words, the increase in the *moment sum* for an increase  $dx$  in the distance of the centre of moments from wheel 1 equals the *load sum* times  $dx$ . If the load sum is constant for an interval  $dx = 1$  foot, as between concentrated loads, the increase of the moment sum for  $dx = 1$  foot equals the corresponding load sum. If the load sum is not constant, but *uniformly* increasing, as when the centre of moments lies within the uniform load, the increase of the moment sum for  $dx = 1$  foot equals the *average* value of the load sum for this one foot interval. The application of the foregoing principles is made clear by the following example.

*Example.*—Give explicit directions for the calculation of a table of load sums and moment sums at intervals of 1 foot from 0' to 400' for Cooper's *E40* loading.

*Solution.*—Calculate the table of load sums by adding

the loads one by one, taking a sub-total for each addition. Thus, the following numbers are added:

1—10  
 4—20's  
 4—13's  
 1—10  
 4—20's  
 4—13's  
 391— 2's

If the final total checks  $284 + 391 \times 2 = 866$ , the table of load sums is correct.

Assume now that the table of load sums for *E40* has been completed. The table of moment sums may now be found as directed below. The following numbers are to be added one by one, taking a sub-total for each addition:

8—10's  
 5—30's  
 5—50's  
 5—70's  
 9—90's  
 5—103's  
 6—116's  
 5—129's  
 8—142's  
 8—152's  
 5—172's  
 5—192's  
 5—212's  
 9—232's  
 5—245's  
 6—258's  
 5—271's  
 5—284's  
 1—285  
 1—287  
 1—289

and all odd numbers up to 865.

If the final total checks up 183,689, which is figured independently, the table of moment sums is correct.

The preceding additions may be made most satisfactorily on a recording adding machine. Table 2 was calculated in this way.

It will be noted that the table of load sums serves as a table of differences for the table of moment sums.

## ARTICLE XI.

### SUMMARY OF FORMULAS.

*Art. 1.*

[illegible]

[illegible]

[illegible]

$$\mathbf{Z} = \mathbf{z}\Sigma\mathbf{w} = \mathbf{z}\mathbf{W} \quad (4)$$

*Art. 2.*

$$\mathbf{Z} = \Sigma \mathbf{w}_n \mathbf{z}_n = \mathbf{C}_a \Sigma \mathbf{w}_n \mathbf{x}_n = \mathbf{C}_a \mathbf{M}_a \quad . \quad . \quad . \quad . \quad (5)$$

$$\frac{dZ}{dx} = C_a W_a = \frac{d(C_a M_a)}{dx} = \frac{C_a dM_a}{dx} \dots \dots (5a)$$

*Art. 3.*

[illegible]

$$S = \Sigma CM \quad (7)$$

$$\frac{dS}{dx} = \Sigma CW \quad (8)$$

*Art. 4. Girder Bridge without Panels.*

### End reactions.

$$R_1 = \frac{M_3 - M_1}{L} - W_1 \dots\dots\dots (9)$$

$$R_2 = W_3 - \frac{M_3 - M_1}{I} \quad . \quad . \quad . \quad . \quad . \quad . \quad (9a)$$

### Bending moment for unequal segments $l_1$ and $l_2$ .

$$M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \quad . \quad . \quad . \quad . \quad . \quad (10)$$

$$\frac{dM}{dx} = \frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2 \quad . \quad . \quad . \quad . \quad . \quad (11)$$

Bending moment at centre.  $l_1 = l_2 = \frac{L}{2}$

$$M = \frac{M_3 + M_1}{2} - M_2 \quad \dots \dots \dots (10a)$$

$$\frac{dM}{dx} = \frac{W_3 + W_1}{2} - W_2 \quad \dots \dots \dots (11a)$$

Shear at any section.

$$S = \frac{M_3 - M_1}{L} - W_2 \quad \dots \dots \dots (12)$$

Location of centre of gravity of loading on span.

$$\bar{x} = \frac{M_3 - M_1 - LW_1}{W_3 - W_1} \quad \dots \dots \dots (13)$$

When  $M_1 = 0$ ,

$$\bar{x} = \frac{M_3}{W_3} \quad \dots \dots \dots (13a)$$

#### Art. 5. Pier Reaction.

For unequal spans  $l_1$  and  $l_2$ .

$$R = \frac{M_3}{l_2} + \frac{M_1}{l_1} - \frac{L}{l_1 l_2} M_2 = \frac{L}{l_1 l_2} \left( \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \right) \quad (14)$$

$$\frac{dR}{dx} = \frac{W_3}{l_2} + \frac{W_1}{l_1} - \frac{L}{l_1 l_2} W_2 = \frac{L}{l_1 l_2} \left( \frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2 \right) \quad (15)$$

For equal spans  $l_1$  and  $l_2$  equal to  $l$ .

$$R = \frac{M_3 + M_1 - 2M_2}{l} \quad \dots \dots \dots (14a)$$

$$\frac{dR}{dx} = \frac{W_3 + W_1 - 2W_2}{l} \quad \dots \dots \dots (15a)$$

Relation between  $R$  and  $M$ ,

$$R = \frac{L}{l_1 l_2} M \quad \dots \dots \dots (16)$$

#### Art. 6. Girder Bridge with Panels.

Shear in end panel; general case.

$$S_a = \frac{1}{L} M_3 + \frac{l_2}{l_1 L} M_1 - \frac{1}{l_1} M_2 = \frac{1}{l_1} \left( \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \right) \quad (17)$$

$$\frac{dS_a}{dx} = \frac{1}{L} W_3 + \frac{l_2}{l_1 L} W_1 - \frac{1}{l_1} W_2 = \frac{1}{l_1} \left( \frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2 \right) \quad (18)$$

Shear in intermediate panel; general case.

$$S_b = \frac{M_4}{L} - \frac{M_3}{p} + \frac{M_2}{p} - \frac{M_1}{L} \quad \dots \quad (19)$$

$$\frac{dS_b}{dx} = \frac{W_4}{L} - \frac{W_3}{p} + \frac{W_2}{p} - \frac{W_1}{L} \quad \dots \quad (20)$$

Shear in intermediate panel; usual case.

$$S = \frac{M_4}{L} - \frac{M_3}{p} = \frac{1}{p} \left( \frac{p}{L} M_4 - M_3 \right) \quad \dots \quad (19a)$$

$$\frac{dS_b}{dx} = \frac{W_4}{L} - \frac{W_3}{p} = \frac{1}{p} \left( \frac{p}{L} W_4 - W_3 \right) \quad \dots \quad (20a)$$

*Art. 7. Through Pratt Truss with Inclined Chord.*

Stress in hanger. Use formulas (14a) and (15a).

Stress in any horizontal chord member; usual case.

$$S_s = \left( \frac{m}{nv} \right) M_4 - \left( \frac{1}{v} \right) M_3 \quad \dots \quad (21)$$

Compression in any inclined top chord member or end post; usual case.

$$S_s = \left( \frac{i}{p} \right) S_5 \quad \dots \quad (22)$$

Compression in vertical post; usual case.

$$S_4 = \left( \frac{a}{bL} \right) M_4 - \left( \frac{1}{p} \right) M_3 \quad \dots \quad (23)$$

Stresses in inclined web members including counters; usual case.

$$S_1, S_2, S_3 = \left( \frac{ta}{cbL} \right) M_4 - \left( \frac{t}{bp} \right) M_3 \quad \dots \quad (24)$$

Stress in inclined counter; special case of loading advanced beyond panel.

$$S_2 = \left(\frac{ta}{cbL}\right)M_1 - \frac{t}{bp}\left(M_3 - \frac{b}{c}M_2\right) = \left(\frac{ta}{cbL}\right)M_1 - \left(\frac{t}{bp}\right)M_3 \quad (25)$$

Counter-tension in vertical post; usual case.

$$T = \left(\frac{d_2 - d_1}{bp}\right)\left(\frac{m}{n}M_1 - M_3\right) = K \cdot M_3 \quad \dots (26)$$

Formulas (21), (23), and (24) are of the general form

$$S = GM_1 - HM_3 \quad \dots \dots \dots (27)$$

where the coefficients  $G$  and  $H$  may be tabulated thus:

Type of member.....	$G$	$H$
Horizontal chord.....	$\frac{m}{nv}$	$\frac{1}{v}$
Vertical post.....	$\frac{a}{bL}$	$\frac{1}{p}$
Inclined web member....	$\frac{ta}{cbL}$	$\frac{t}{bp}$

The rate of variation of  $S$  in formula (27) is

$$\frac{dS}{dx} = GW_1 - HW_3 = H\left(\frac{G}{H}W_1 - W_3\right) \quad \dots (28)$$

When  $S$  in formulas (21), (23), or (24) is a maximum

$$\left(\frac{G}{H}W_1 - W_3\right) \text{ passes through zero.}$$

*Through Pratt Truss—Parallel Chords.*

Stress in hanger,—use formulas (14a) and (15a)

$$\text{Stress in horizontal chord} = S_3 = \left(\frac{m}{nv}\right)M_1 - \left(\frac{1}{v}\right)M_3 \quad (21)$$

$$\text{“ “ vertical post} = S_4 = \left(\frac{1}{L}\right)M_1 - \left(\frac{1}{p}\right)M_3 \quad \dots (29)$$

$$\text{“ “ inclined web} = S_1 = \left(\frac{t}{cL}\right)M_1 - \left(\frac{t}{cp}\right)M_3 = \frac{t}{c}S_4 \quad (30)$$

$$\text{Stress in end post} = S_6 = \frac{-S_5}{p} \quad \dots \quad (22)$$

Formulas (21), (29), and (30) are of the general form

$$S = G \cdot M_4 - H \cdot M_3 \quad \dots \quad (27)$$

and their rate of variation is

$$\frac{dS}{dx} = H \left( \frac{G}{H} W_4 - W_3 \right) \quad \dots \quad (28)$$

$G$  and  $H$  are the coefficients of  $M_4$  and  $M_3$  in equations (21), (29), and (30), respectively.

When  $S$  in formulas (21), (29), or (30) is a maximum,  $\left( \frac{G}{H} W_4 - W_3 \right)$  passes through zero.

*Art. 9. Equivalent Uniform Loads.*

$$q = \frac{2M}{l_1 l_2} = \frac{2R}{L} \quad \dots \quad (31)$$

$$M = q \left( \frac{l_1 l_2}{2} \right) \quad \dots \quad (32)$$

$$R = q \left( \frac{L}{2} \right) = q \left( \frac{l_1 + l_2}{2} \right) \quad \dots \quad (33)$$





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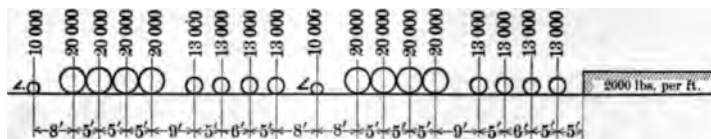
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TABLE 1

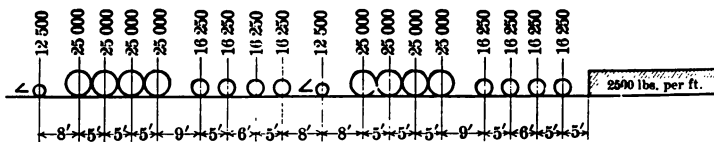
## STANDARD LOADINGS

Loads given are for one rail.

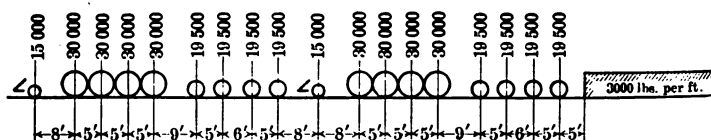
## COOPER'S E 40:



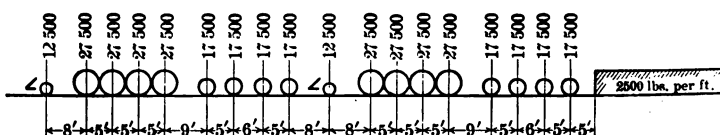
## COOPER'S E 50:



## COOPER'S E 60:



## COMMON STANDARD-1904-PACIFIC SYSTEM



## D. L. &amp; W. R. R.:

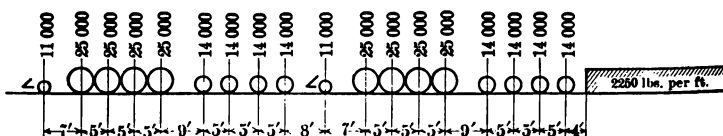


TABLE 2

LOAD SUMS AND MOMENT SUMS FOR COOPER'S  
AND OTHER STANDARD LOADINGS

NOTE.—Load Sums and Moment Sums are given per rail in thousands of pounds and foot-pounds respectively.

## LIVE-LOAD STRESSES

COOPER'S E40. 0'-50'					COOPER'S E40. 50'-100'				
Length	Wheel	Load	Load Sum	Moment Sum	Length	Wheel	Load	Load Sum	Moment Sum
0	w. 1	10	10	0	50	.....	..	...	3780
1	.....	..	..	10	51	.....	..	...	3922
2	.....	..	..	20	52	.....	..	...	4064
3	.....	..	..	30	53	.....	..	...	4206
4	.....	..	..	40	54	.....	..	...	4348
5	.....	..	..	50	55	.....	..	...	4490
6	.....	..	..	60	56	w. 10	10	152	4632
7	.....	..	..	70	57	.....	..	...	4784
8	w. 2	20	30	80	58	.....	..	...	4936
9	.....	..	..	110	59	.....	..	...	5088
10	.....	..	..	140	60	.....	..	...	5240
11	.....	..	..	170	61	.....	..	...	5392
12	.....	..	..	200	62	.....	..	...	5544
13	w. 3	20	30	230	63	.....	..	...	5696
14	.....	..	..	260	64	w. 11	20	172	5848
15	.....	..	..	290	65	.....	..	...	6020
16	.....	..	..	320	66	.....	..	...	6192
17	.....	..	..	350	67	.....	..	...	6364
18	w. 4	20	70	420	68	w. 12	20	192	6536
19	.....	..	..	550	69	.....	..	...	6708
20	.....	..	..	680	70	.....	..	...	6900
21	.....	..	..	810	71	.....	..	...	7092
22	.....	..	..	940	72	.....	..	...	7284
23	w. 5	20	30	1070	73	.....	..	...	7476
24	.....	..	..	1200	74	w. 13	20	212	7668
25	.....	..	..	1330	75	.....	..	...	7880
26	.....	..	..	1460	76	.....	..	...	8092
27	.....	..	..	1590	77	.....	..	...	8304
28	.....	..	..	1720	78	w. 14	20	232	8516
29	.....	..	..	1850	79	.....	..	...	8728
30	.....	..	..	1980	80	.....	..	...	8960
31	.....	..	..	2110	81	.....	..	...	9192
32	w. 6	13	103	2240	82	.....	..	...	9424
33	.....	..	..	2370	83	.....	..	...	9656
34	.....	..	..	2500	84	.....	..	...	9888
35	.....	..	..	2630	85	.....	..	...	10120
36	.....	..	..	2760	86	.....	..	...	10352
37	w. 7	13	116	2890	87	.....	..	...	10584
38	.....	..	..	3020	88	w. 15	13	245	10816
39	.....	..	..	3150	89	.....	..	...	11061
40	.....	..	..	3280	90	.....	..	...	11306
41	.....	..	..	3410	91	.....	..	...	11551
42	.....	..	..	3540	92	.....	..	...	11796
43	w. 8	13	129	3670	93	w. 16	13	258	12041
44	.....	..	..	3800	94	.....	..	...	12299
45	.....	..	..	3930	95	.....	..	...	12557
46	.....	..	..	4060	96	.....	..	...	12815
47	.....	..	..	4190	97	.....	..	...	13073
48	w. 9	13	142	4320	98	.....	..	...	13331
49	.....	..	..	4450	99	w. 17	13	271	13589
50	.....	..	..	4580	100	.....	..	...	13860

## LIVE-LOAD STRESSES

69

COOPER'S E40. 100'-150'

COOPER'S E40. 150'-200'

Length	Wheel	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
100				13860	150		366	29689
101				14131	151		368	30056
102				14402	152		370	30425
103				14673	153		372	30796
104	w. 18	13	234	14944	154		374	31169
105				15228	155		376	31544
106				15512	156		378	31921
107				15796	157		380	32300
108				16080	158		382	32681
109			284	16364	159		384	33064
110			286	16649	160		386	33449
111			288	16936	161		388	33836
112			290	17225	162		390	34225
113			292	17516	163		392	34616
114			294	17809	164		394	35009
115			296	18104	165		396	35404
116			298	18401	166		398	35801
117			300	18700	167		400	36200
118			302	19001	168		402	36601
119			304	19304	169		404	37004
120		Uniform Load = 2,000 pounds per foot	306	19609	170	Uniform Load = 2,000 pounds per foot	406	37409
121			308	19916	171		408	37816
122			310	20225	172		410	38225
123			312	20536	173		412	38636
124			314	20849	174		414	39049
125			316	21164	175		416	39464
126			318	21481	176		418	39881
127			320	21800	177		420	40300
128			322	22121	178		422	40721
129			324	22444	179		424	41144
130		Uniform Load = 2,000 pounds per foot	326	22769	180	Uniform Load = 2,000 pounds per foot	426	41569
131			328	23096	181		428	41996
132			330	23425	182		430	42425
133			332	23756	183		432	42856
134			334	24089	184		434	43289
135			336	24424	185		436	43724
136			338	24761	186		438	44161
137			340	25100	187		440	44600
138			342	25441	188		442	45041
139			344	25784	189		444	45484
140		Uniform Load = 2,000 pounds per foot	346	26129	190	Uniform Load = 2,000 pounds per foot	446	45929
141			348	26476	191		448	46376
142			350	26825	192		450	46825
143			352	27176	193		452	47276
144			354	27529	194		454	47729
145			356	27884	195		456	48184
146			358	28241	196		458	48641
147			360	28600	197		460	49100
148			362	28961	198		462	49561
149			364	29324	199		464	50024
150			366	29689	200		466	50489

[illegible]

## LIVE-LOAD STRESSES

71

COOPER'S E40. 300'-350'

COOPER'S E40. 350'-400'

Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
300	Uniform Load = 2,000 pounds per foot	666	107089	350	Uniform Load = 2,000 pounds per foot	766	142889
301		668	107756	351		768	143656
302		670	108425	352		770	144425
303		672	109096	353		772	145196
304		674	109769	354		774	145969
305		676	110444	355		776	146744
306		678	111121	356		778	147521
307		680	111800	357		780	148300
308		682	112481	358		782	149081
309		684	113164	359		784	149864
310		686	113849	360		786	150649
311		688	114536	361		788	151436
312		690	115225	362		790	152225
313		692	115916	363		792	153016
314		694	116609	364		794	153809
315		696	117304	365		796	154604
316		698	118001	366		798	155401
317		700	118700	367		800	156200
318		702	119401	368		802	157001
319		704	120104	369		804	157804
320		706	120809	370		806	158609
321		708	121516	371		808	159416
322		710	122225	372		810	160225
323		712	122936	373		812	161036
324		714	123649	374		814	161849
325		716	124364	375		816	162664
326		718	125081	376		818	163481
327		720	125800	377		820	164300
328		722	126521	378		822	165121
329		724	127244	379		824	165944
330		726	127969	380		826	166769
331		728	128696	381		828	167596
332		730	129425	382		830	168425
333		732	130156	383		832	169256
334		734	130889	384		834	170089
335		736	131624	385		836	170924
336		738	132361	386		838	171761
337		740	133100	387		840	172600
338		742	133841	388		842	173441
339		744	134584	389		844	174284
340		746	135329	390		846	175129
341		748	136076	391		848	175976
342		750	136825	392		850	176825
343		752	137576	393		852	177676
344		754	138329	394		854	178529
345		756	139084	395		856	179384
346		758	139841	396		858	180241
347		760	140600	397		860	181100
348		762	141361	398		862	181961
349		764	142124	399		864	182824
350		766	142889	400		866	183689



## LIVE-LOAD STRESSES

COOPER'S E50. 0'-50'					COOPER'S E50. 50'-100'				
Length	Wheel	Load	Load Sums	Moment Sums	Length	Wheel	Load	Load Sums	Moment Sums
0	w. 1	12.50	12.50	00.00	50	.....	.....	.....	4725.00
1	.....	.....	.....	12.50	51	.....	.....	.....	4902.50
2	.....	.....	.....	25.00	52	.....	.....	.....	5080.00
3	.....	.....	.....	37.50	53	.....	.....	.....	5257.50
4	.....	.....	.....	50.00	54	.....	.....	.....	5435.00
5	.....	.....	.....	62.50	55	.....	.....	.....	5612.50
6	.....	.....	.....	75.00	56	w. 10	12.50	190.00	5790.00
7	.....	.....	.....	87.50	57	.....	.....	.....	5980.00
8	w. 2	25.00	37.50	100.00	58	.....	.....	.....	6170.00
9	.....	.....	.....	137.50	59	.....	.....	.....	6360.00
10	.....	.....	.....	175.00	60	.....	.....	.....	6550.00
11	.....	.....	.....	212.50	61	.....	.....	.....	6740.00
12	.....	.....	.....	250.00	62	.....	.....	.....	6930.00
13	w. 3	25.00	62.50	287.50	63	.....	.....	.....	7120.00
14	.....	.....	.....	350.00	64	w. 11	25.00	215.00	7310.00
15	.....	.....	.....	412.50	65	.....	.....	.....	7525.00
16	.....	.....	.....	475.00	66	.....	.....	.....	7740.00
17	.....	.....	.....	537.50	67	.....	.....	.....	7955.00
18	w. 4	25.00	87.50	600.00	68	.....	.....	.....	8170.00
19	.....	.....	.....	687.50	69	w. 12	25.00	240.00	8385.00
20	.....	.....	.....	775.00	70	.....	.....	.....	8625.00
21	.....	.....	.....	862.50	71	.....	.....	.....	8865.00
22	.....	.....	.....	950.00	72	.....	.....	.....	9105.00
23	w. 5	25.00	112.50	1037.50	73	.....	.....	.....	9345.00
24	.....	.....	.....	1150.00	74	w. 13	25.00	265.00	9585.00
25	.....	.....	.....	1262.50	75	.....	.....	.....	9850.00
26	.....	.....	.....	1375.00	76	.....	.....	.....	10115.00
27	.....	.....	.....	1487.50	77	.....	.....	.....	10380.00
28	.....	.....	.....	1600.00	78	.....	.....	.....	10645.00
29	.....	.....	.....	1712.50	79	w. 14	25.00	290.00	10910.00
30	.....	.....	.....	1825.00	80	.....	.....	.....	11200.00
31	.....	.....	.....	1937.50	81	.....	.....	.....	11490.00
32	w. 6	16.25	128.75	2050.00	82	.....	.....	.....	11780.00
33	.....	.....	.....	2178.75	83	.....	.....	.....	12070.00
34	.....	.....	.....	2307.50	84	.....	.....	.....	12360.00
35	.....	.....	.....	2436.25	85	.....	.....	.....	12650.00
36	.....	.....	.....	2565.00	86	.....	.....	.....	12940.00
37	w. 7	16.25	145.00	2693.75	87	.....	.....	.....	13230.00
38	.....	.....	.....	2838.75	88	w. 15	16.25	306.25	13520.00
39	.....	.....	.....	2983.75	89	.....	.....	.....	13826.25
40	.....	.....	.....	3128.75	90	.....	.....	.....	14132.50
41	.....	.....	.....	3273.75	91	.....	.....	.....	14438.75
42	.....	.....	.....	3418.75	92	.....	.....	.....	14745.00
43	w. 8	16.25	161.25	3563.75	93	w. 16	16.25	322.50	15051.25
44	.....	.....	.....	3725.00	94	.....	.....	.....	15373.75
45	.....	.....	.....	3886.25	95	.....	.....	.....	15696.25
46	.....	.....	.....	4047.50	96	.....	.....	.....	16018.75
47	.....	.....	.....	4208.75	97	.....	.....	.....	16341.25
48	w. 9	16.25	177.50	4370.00	98	.....	.....	.....	16663.75
49	.....	.....	.....	4547.50	99	w. 17	16.25	338.75	16986.25
50	.....	.....	.....	4725.00	100	.....	.....	.....	17325.00

## LIVE-LOAD STRESSES

73

COOPER'S E50. 100'-150'

COOPER'S E50. 150'-200'

Length	Wheel	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
100	.....	.....	.....	17325.00	150	.....	457.50	37111.25
101	.....	.....	.....	17663.75	151	.....	460.00	37570.00
102	.....	.....	.....	18002.50	152	.....	462.50	38031.25
103	.....	.....	.....	18341.25	153	.....	465.00	38495.00
104	w. 18	16.25	355.00	18680.00	154	.....	467.50	38961.25
105	.....	.....	.....	19035.00	155	.....	470.00	39430.00
106	.....	.....	.....	19390.00	156	.....	472.50	39901.25
107	.....	.....	.....	19745.00	157	.....	475.00	40375.00
108	.....	.....	.....	20100.00	158	.....	477.50	40851.25
109	.....	.....	355.00	20455.00	159	.....	480.00	41330.00
110	.....	Uniform Load = 2,500 pounds per foot	357.50	20811.25	160	.....	482.50	41811.25
111	.....		360.00	21170.00	161	.....	485.00	42295.00
112	.....		362.50	21531.25	162	.....	487.50	42781.25
113	.....		365.00	21895.00	163	.....	490.00	43270.00
114	.....		367.50	22261.25	164	.....	492.50	43761.25
115	.....		370.00	22630.00	165	.....	495.00	44255.00
116	.....		372.50	23001.25	166	.....	497.50	44751.25
117	.....		375.00	23375.00	167	.....	500.00	45250.00
118	.....		377.50	23751.25	168	.....	502.50	45751.25
119	.....		380.00	24130.00	169	.....	505.00	46255.00
120	.....		382.50	24511.25	170	.....	507.50	46761.25
121	.....		385.00	24895.00	171	.....	510.00	47270.00
122	.....		387.50	25281.25	172	.....	512.50	47781.25
123	.....		390.00	25670.00	173	.....	515.00	48295.00
124	.....		392.50	26061.25	174	.....	517.50	48811.25
125	.....		395.00	26455.00	175	.....	520.00	49330.00
126	.....		397.50	26851.25	176	.....	522.50	49851.25
127	.....		400.00	27250.00	177	.....	525.00	50375.00
128	.....		402.50	27651.25	178	.....	527.50	50901.25
129	.....		405.00	28055.00	179	.....	530.00	51430.00
130	.....		407.50	28461.25	180	.....	532.50	51961.25
131	.....		410.00	28870.00	181	.....	535.00	52495.00
132	.....		412.50	29281.25	182	.....	537.50	53031.25
133	.....		415.00	29695.00	183	.....	540.00	53570.00
134	.....		417.50	30111.25	184	.....	542.50	54111.25
135	.....		420.00	30530.00	185	.....	545.00	54655.00
136	.....		422.50	30951.25	186	.....	547.50	55201.25
137	.....		425.00	31375.00	187	.....	550.00	55750.00
138	.....		427.50	31801.25	188	.....	552.50	56301.25
139	.....		430.00	32230.00	189	.....	555.00	56855.00
140	.....	Uniform Load = 2,500 pounds per foot	432.50	32661.25	190	.....	557.50	57411.25
141	.....		435.00	33095.00	191	.....	560.00	57970.00
142	.....		437.50	33531.25	192	.....	562.50	58531.25
143	.....		440.00	33970.00	193	.....	565.00	59095.00
144	.....		442.50	34411.00	194	.....	567.50	59661.25
145	.....		445.00	34855.00	195	.....	570.00	60230.00
146	.....		447.50	35301.25	196	.....	572.50	60801.25
147	.....		450.00	35750.00	197	.....	575.00	61375.00
148	.....		452.50	36201.25	198	.....	577.50	61951.25
149	.....		455.00	36655.00	199	.....	580.00	62530.00
150	.....		457.50	37111.25	200	.....	582.50	63111.25

COOPER'S E50. 200'-250'				COOPER'S E50. 250'-300'			
Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
200	Uniform Load = 2,500 pounds per foot	582.50	63111.25	250	Uniform Load = 2,500 pounds per foot	707.50	95361.25
201		585.00	63695.00	251		710.00	96070.00
202		587.50	64281.25	252		712.50	96781.25
203		590.00	64870.00	253		715.00	97495.00
204		592.50	65461.25	254		717.50	98211.25
205		595.00	66055.00	255		720.00	98930.00
206		597.50	66651.25	256		722.50	99651.25
207		600.00	67250.00	257		725.00	100375.00
208		602.50	67851.25	258		727.50	101101.25
209		605.00	68455.00	259		730.00	101830.00
210		607.50	69061.25	260		732.50	102561.25
211		610.00	69670.00	261		735.00	103295.00
212		612.50	70281.25	262		737.50	104031.25
213		615.00	70895.00	263		740.00	104770.00
214		617.50	71511.25	264		742.50	105511.25
215		620.00	72130.00	265		745.00	106255.00
216		622.50	72751.25	266		747.50	107001.25
217		625.00	73375.00	267		750.00	107750.00
218		627.50	74001.25	268		752.50	108501.25
219		630.00	74630.00	269		755.00	109255.00
220		632.50	75261.25	270		757.50	110011.25
221		635.00	75895.00	271		760.00	110770.00
222		637.50	76531.25	272		762.50	111531.25
223		640.00	77170.00	273		765.00	112295.00
224		642.50	77811.25	274		767.50	113061.25
225		645.00	78455.00	275		770.00	113830.00
226		647.50	79101.25	276		772.50	114601.25
227		650.00	79750.00	277		775.00	115375.00
228		652.50	80401.25	278		777.50	116151.25
229		655.00	81055.00	279		780.00	116930.00
230		657.50	81711.25	280		782.50	117711.25
231		660.00	82370.00	281		785.00	118495.00
232		662.50	83031.25	282		787.50	119281.25
233		665.00	83695.00	283		790.00	120070.00
234		667.50	84361.25	284		792.50	120861.25
235		670.00	85030.00	285		795.00	121655.00
236		672.50	85701.25	286		797.50	122451.25
237		675.00	86375.00	287		800.00	123250.00
238		677.50	87051.25	288		802.50	124051.25
239		680.00	87730.00	289		805.00	124855.00
240		682.50	88411.25	290		807.50	125661.25
241		685.00	89095.00	291		810.00	126470.00
242		687.50	89781.25	292		812.50	127281.25
243		690.00	90470.00	293		815.00	128095.00
244		692.50	91161.25	294		817.50	128911.25
245		695.00	91855.00	295		820.00	129730.00
246		697.50	92551.25	296		822.50	130551.25
247		700.00	93250.00	297		825.00	131375.00
248		702.50	93951.25	298		827.50	132201.25
249		705.00	94655.00	299		830.00	133030.00
250		707.50	95361.25	300		832.50	133861.25

## LIVE-LOAD STRESSES

75

COOPER'S E50. 300'-350'				COOPER'S E50. 350'-400'			
Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
300	Uniform Load = 2,500 pounds per foot	832.50	133861.25	350	Uniform Load = 2,500 pounds per foot	957.50	178611.25
301		835.00	134695.00	351		960.00	179570.00
302		837.50	135531.25	352		962.50	180531.25
303		840.00	136370.00	353		965.00	181495.00
304		842.50	137211.25	354		967.50	182461.25
305		845.00	138055.00	355		970.00	183430.00
306		847.50	138901.25	356		972.50	184401.25
307		850.00	139750.00	357		975.00	185375.00
308		852.50	140601.25	358		977.50	186351.25
309		855.00	141455.00	359		980.00	187330.00
310		857.50	142311.25	360		982.50	188311.25
311		860.00	143170.00	361		985.00	189295.00
312		862.50	144031.25	362		987.50	190281.25
313		865.00	144895.00	363		990.00	191270.00
314		867.50	145761.25	364		992.50	192261.25
315		870.00	146630.00	365		995.00	193255.00
316		872.50	147501.25	366		997.50	194251.25
317		875.00	148375.00	367		1000.00	195250.00
318		877.50	149251.25	368		1002.50	196251.25
319		880.00	150130.00	369		1005.00	197255.00
320		882.50	151011.25	370		1007.50	198261.25
321		885.00	151895.00	371		1010.00	199270.00
322		887.50	152781.25	372		1012.50	200281.25
323		890.00	153670.00	373		1015.00	201295.00
324		892.50	154561.25	374		1017.50	202311.25
325		895.00	155455.00	375		1020.00	203330.00
326		897.50	156351.25	376		1022.50	204351.25
327		900.00	157250.00	377		1025.00	205375.00
328		902.50	158151.25	378		1027.50	206401.25
329		905.00	159055.00	379		1030.00	207430.00
330		907.50	159961.25	380		1032.50	208461.25
331		910.00	160870.00	381		1035.00	209495.00
332		912.50	161781.25	382		1037.50	210531.25
333		915.00	162695.00	383		1040.00	211570.00
334		917.50	163611.25	384		1042.50	212611.25
335		920.00	164530.00	385		1045.00	213655.00
336		922.50	165451.25	386		1047.50	214701.25
337		925.00	166375.00	387		1050.00	215750.00
338		927.50	167301.25	388		1052.50	216801.25
339		930.00	168230.00	389		1055.00	217855.00
340		932.50	169161.25	390		1057.50	218911.25
341		935.00	170095.00	391		1060.00	219970.00
342		937.50	171031.25	392		1062.50	221031.25
343		940.00	171970.00	393		1065.00	222095.00
344		942.50	172911.25	394		1067.50	223161.25
345		945.00	173855.00	395		1070.00	224230.00
346		947.50	174801.25	396		1072.50	225301.25
347		950.00	175750.00	397		1075.00	226375.00
348		952.50	176701.25	398		1077.50	227451.25
349		955.00	177655.00	399		1080.00	228530.00
350		957.50	178611.25	400		1082.50	229611.25

## LIVE-LOAD STRESSES

COOPER'S E60. 0'-50'

COOPER'S E60. 50'-100'

Length	Wheel	Load	Load Sums	Moment Sums	Length	Wheel	Load	Load Sums	Moment Sums
0	w. 1	15.0	15.0	00.00	50	.....	.....	.....	5670.00
1	.....	.....	.....	15.00	51	.....	.....	.....	5883.00
2	.....	.....	.....	30.00	52	.....	.....	.....	6096.00
3	.....	.....	.....	45.00	53	.....	.....	.....	6309.00
4	.....	.....	.....	60.00	54	.....	.....	.....	6522.00
5	.....	.....	.....	75.00	55	.....	.....	.....	6735.00
6	.....	.....	.....	90.00	56	w. 10	15.0	228.0	6948.00
7	.....	.....	.....	105.00	57	.....	.....	.....	7176.00
8	w. 2	30.0	45.0	120.00	58	.....	.....	.....	7404.00
9	.....	.....	.....	165.00	59	.....	.....	.....	7632.00
10	.....	.....	.....	210.00	60	.....	.....	.....	7860.00
11	.....	.....	.....	255.00	61	.....	.....	.....	8088.00
12	.....	.....	.....	300.00	62	.....	.....	.....	8316.00
13	w. 3	30.0	75.0	345.00	63	.....	.....	.....	8544.00
14	.....	.....	.....	420.00	64	w. 11	30.0	258.0	8772.00
15	.....	.....	.....	495.00	65	.....	.....	.....	9030.00
16	.....	.....	.....	570.00	66	.....	.....	.....	9288.00
17	.....	.....	.....	645.00	67	.....	.....	.....	9546.00
18	w. 4	30.0	105.0	720.00	68	.....	.....	.....	9804.00
19	.....	.....	.....	825.00	69	w. 12	30.0	288.0	10062.00
20	.....	.....	.....	930.00	70	.....	.....	.....	10350.00
21	.....	.....	.....	1035.00	71	.....	.....	.....	10638.00
22	.....	.....	.....	1140.00	72	.....	.....	.....	10926.00
23	w. 5	30.0	135.0	1245.00	73	.....	.....	.....	11214.00
24	.....	.....	.....	1380.00	74	w. 13	30.0	318.0	11502.00
25	.....	.....	.....	1515.00	75	.....	.....	.....	11820.00
26	.....	.....	.....	1650.00	76	.....	.....	.....	12138.00
27	.....	.....	.....	1785.00	77	.....	.....	.....	12456.00
28	.....	.....	.....	1920.00	78	.....	.....	.....	12774.00
29	.....	.....	.....	2055.00	79	w. 14	30.0	348.0	13092.00
30	.....	.....	.....	2190.00	80	.....	.....	.....	13440.00
31	.....	.....	.....	2325.00	81	.....	.....	.....	13788.00
32	w. 6	19.5	154.5	2460.00	82	.....	.....	.....	14136.00
33	.....	.....	.....	2614.50	83	.....	.....	.....	14484.00
34	.....	.....	.....	2769.00	84	.....	.....	.....	14832.00
35	.....	.....	.....	2923.50	85	.....	.....	.....	15180.00
36	.....	.....	.....	3078.00	86	.....	.....	.....	15528.00
37	w. 7	19.5	174.0	3232.50	87	.....	.....	.....	15876.00
38	.....	.....	.....	3406.50	88	w. 15	19.5	367.5	16224.00
39	.....	.....	.....	3580.50	89	.....	.....	.....	16591.00
40	.....	.....	.....	3754.50	90	.....	.....	.....	16959.00
41	.....	.....	.....	3928.50	91	.....	.....	.....	17326.50
42	.....	.....	.....	4102.50	92	.....	.....	.....	17694.00
43	w. 8	19.5	193.5	4276.50	93	w. 16	19.5	387.0	18061.50
44	.....	.....	.....	4470.00	94	.....	.....	.....	18448.00
45	.....	.....	.....	4663.50	95	.....	.....	.....	18835.50
46	.....	.....	.....	4857.00	96	.....	.....	.....	19222.50
47	.....	.....	.....	5050.50	97	.....	.....	.....	19609.50
48	w. 9	19.5	213.0	5244.00	98	.....	.....	.....	19996.50
49	.....	.....	.....	5457.00	99	w. 17	19.5	406.5	20383.50
50	.....	.....	.....	5670.00	100	.....	.....	.....	20790.00

## LIVE-LOAD STRESSES

77

COOPER'S E60. 100'-150'

COOPER'S E60. 150'-200'

Length	Wheel	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
100	.....	.....	.....	20790.00	150	.....	549.0	44533.50
101	.....	.....	.....	21196.50	151	.....	552.0	45084.00
102	.....	.....	.....	21603.00	152	.....	555.0	45637.50
103	.....	.....	.....	22009.50	153	.....	558.0	46194.00
104	w. 18	19.5	426.0	22416.00	154	.....	561.0	46753.50
105	.....	.....	.....	22842.00	155	.....	564.0	47316.00
106	.....	.....	.....	23268.00	156	.....	567.0	47881.50
107	.....	.....	.....	23694.00	157	.....	570.0	48450.00
108	.....	.....	.....	24120.00	158	.....	573.0	49021.50
109	.....	.....	426.0	24546.00	159	.....	576.0	49596.00
110	Uniform Load = 3,000 pounds per foot	.....	429.0	24973.50	160	Uniform Load = 3,000 pounds per foot	579.0	50173.50
111		.....	432.0	25404.00	161		582.0	50754.00
112		.....	435.0	25837.50	162		585.0	51337.50
113		.....	438.0	26274.00	163		588.0	51924.00
114		.....	441.0	26713.50	164		591.0	52513.50
115		.....	444.0	27156.00	165		594.0	53106.00
116		.....	447.0	27601.50	166		597.0	53701.50
117		.....	450.0	28050.00	167		600.0	54300.00
118		.....	453.0	28501.50	168		603.0	54901.50
119		.....	456.0	28956.00	169		606.0	55506.00
120		.....	459.0	29413.50	170		609.0	56113.50
121		.....	462.0	29874.00	171		612.0	56724.00
122		.....	465.0	30337.50	172		615.0	57337.50
123		.....	468.0	30804.00	173		618.0	57954.00
124		.....	471.0	31273.50	174		621.0	58573.50
125		.....	474.0	31746.00	175		624.0	59196.00
126		.....	477.0	32221.50	176		627.0	59821.50
127		.....	480.0	32700.00	177		630.0	60450.00
128		.....	483.0	33181.50	178		633.0	61081.50
129		.....	486.0	33666.00	179		636.0	61716.00
130		.....	489.0	34153.50	180		639.0	62353.50
131		.....	492.0	34644.00	181		642.0	62994.00
132		.....	495.0	35137.50	182		645.0	63637.50
133		.....	498.0	35634.00	183		648.0	64284.00
134		.....	501.0	36133.50	184		651.0	64933.50
135		.....	504.0	36636.00	185		654.0	65586.00
136		.....	507.0	37141.50	186		657.0	66241.50
137		.....	510.0	37650.00	187		660.0	66900.00
138		.....	513.0	38161.50	188		663.0	67561.50
139		.....	516.0	38676.00	189		666.0	68226.00
140		.....	519.0	39193.50	190		669.0	68893.50
141		.....	522.0	39714.00	191		672.0	69564.00
142		.....	525.0	40237.50	192		675.0	70237.50
143		.....	528.0	40764.00	193		678.0	70914.00
144		.....	531.0	41293.50	194		681.0	71593.50
145		.....	534.0	41826.00	195		684.0	72276.00
146		.....	537.0	42361.50	196		687.0	72961.50
147		.....	540.0	42900.00	197		690.0	73650.00
148		.....	543.0	43441.50	198		693.0	74341.50
149		.....	546.0	43986.00	199		696.0	75036.00
150		.....	549.0	44533.50	200		699.0	75733.50

## LIVE-LOAD STRESSES

COOPER'S E60. 200'-250'

COOPER'S E60. 250'-300'

Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
200		699.0	75733.50	250		849.0	114433.50
201		702.0	76434.00	251		852.0	115284.00
202		705.0	77137.50	252		855.0	116137.50
203		708.0	77844.00	253		858.0	116994.00
204		711.0	78553.50	254		861.0	117853.50
205		714.0	79266.00	255		864.0	118716.00
206		717.0	79981.50	256		867.0	119581.50
207		720.0	80700.00	257		870.0	120450.00
208		723.0	81421.50	258		873.0	121321.50
209		726.0	82146.00	259		876.0	122196.00
210		729.0	82873.50	260		879.0	123073.50
211		732.0	83604.00	261		882.0	123954.00
212		735.0	84337.50	262		885.0	124837.50
213		738.0	85074.00	263		888.0	125724.00
214		741.0	85813.50	264		891.0	126613.50
215		744.0	86556.00	265		894.0	127506.00
216		747.0	87301.50	266		897.0	128401.50
217		750.0	88050.00	267		900.0	129300.00
218		753.0	88801.50	268		903.0	130201.50
219		756.0	89556.00	269		906.0	131106.00
220		759.0	90313.50	270		909.0	132013.50
221		762.0	91074.00	271		912.0	132924.00
222		765.0	91837.50	272		915.0	133837.50
223		768.0	92604.00	273		918.0	134754.00
224		771.0	93373.50	274		921.0	135673.50
225		774.0	94146.00	275		924.0	136596.00
226		777.0	94921.50	276		927.0	137521.50
227		780.0	95700.00	277		930.0	138450.00
228		783.0	96481.50	278		933.0	139381.50
229		786.0	97266.00	279		936.0	140316.00
230		789.0	98053.50	280		939.0	141253.50
231		792.0	98844.00	281		942.0	142194.00
232		795.0	99637.50	282		945.0	143137.50
233		798.0	100434.00	283		948.0	144084.00
234		801.0	101233.50	284		951.0	145033.50
235		804.0	102036.00	285		954.0	145986.00
236		807.0	102841.50	286		957.0	146941.50
237		810.0	103650.00	287		960.0	147900.00
238		813.0	104461.50	288		963.0	148861.50
239		816.0	105276.00	289		966.0	149826.00
240		819.0	106093.50	290		969.0	150793.50
241		822.0	106914.00	291		972.0	151764.00
242		825.0	107737.50	292		975.0	152737.50
243		828.0	108564.00	293		978.0	153714.00
244		831.0	109393.50	294		981.0	154693.50
245		834.0	110226.00	295		984.0	155676.00
246		837.0	111061.50	296		987.0	156661.50
247		840.0	111900.00	297		990.0	157650.00
248		843.0	112741.50	298		993.0	158641.50
249		846.0	113586.00	299		996.0	159636.00
250		849.0	114433.50	300		999.0	160633.50

Uniform Load = 3,000 pounds per foot				Uniform Load = 3,000 pounds per foot			
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## LIVE-LOAD STRESSES

79

COOPER'S E60. 300'-350'

COOPER'S E60. 350'-400'

Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
300	Uniform Load = 3,000 pounds per foot	999.0	160633.50	350	Uniform Load = 3,000 pounds per foot	1149.0	214333.50
301		1002.0	161634.00	351		1152.0	215484.00
302		1005.0	162637.50	352		1155.0	216637.50
303		1008.0	163644.00	353		1158.0	217794.00
304		1011.0	164653.50	354		1161.0	218953.50
305		1014.0	165666.00	355		1164.0	220116.00
306		1017.0	166681.50	356		1167.0	221281.50
307		1020.0	167700.00	357		1170.0	222450.00
308		1023.0	168721.50	358		1173.0	223621.50
309		1026.0	169746.00	359		1176.0	224796.00
310		1029.0	170773.50	360		1179.0	225973.50
311		1032.0	171804.00	361		1182.0	227154.00
312		1035.0	172837.50	362		1185.0	228337.50
313		1038.0	173874.00	363		1188.0	229524.00
314		1041.0	174913.50	364		1191.0	230713.50
315		1044.0	175956.00	365		1194.0	231906.00
316		1047.0	177001.50	366		1197.0	233101.50
317		1050.0	178050.00	367		1200.0	234300.00
318		1053.0	179101.50	368		1203.0	235501.50
319		1056.0	180156.00	369		1206.0	236706.00
320		1059.0	181213.50	370		1209.0	237913.50
321		1062.0	182274.00	371		1212.0	239124.00
322		1065.0	183337.50	372		1215.0	240337.50
323		1038.0	184404.00	373		1218.0	241554.00
324		1071.0	185473.50	374		1221.0	242773.50
325		1074.0	186546.00	375		1224.0	243996.00
326		1077.0	187621.50	376		1227.0	245221.50
327		1080.0	188700.00	377		1230.0	246450.00
328		1083.0	189781.50	378		1233.0	247681.50
329		1086.0	190866.00	379		1236.0	248916.00
330		1089.0	191953.50	380		1239.0	250153.50
331		1092.0	193044.00	381		1242.0	251394.00
332		1095.0	194137.50	382		1245.0	252637.50
333		1098.0	195234.00	383		1248.0	253884.00
334		1101.0	196333.50	384		1251.0	255133.50
335		1104.0	197436.00	385		1254.0	256386.00
336		1107.0	198541.50	386		1257.0	257641.50
337		1110.0	199650.00	387		1260.0	258900.00
338		1113.0	200761.50	388		1263.0	260161.50
339		1116.0	201876.00	389		1266.0	261426.00
340		1119.0	202993.50	390		1269.0	262693.50
341		1122.0	204114.00	391		1272.0	263964.00
342		1125.0	205237.50	392		1275.0	265237.50
343		1128.0	206364.00	393		1278.0	266514.00
344		1131.0	207493.50	394		1281.0	267793.50
345		1134.0	208626.00	395		1284.0	269076.00
346		1137.0	209761.50	396		1287.0	270361.50
347		1140.0	210900.00	397		1290.0	271650.00
348		1143.0	212041.50	398		1293.0	272941.50
349		1146.0	213186.00	399		1296.0	274236.00
350		1149.0	214333.50	400		1299.0	275533.50



## LIVE-LOAD STRESSES

COMMON STANDARD 0'-50'

COMMON STANDARD 50'-100'

Length	Wheel	Load	Load Sums	Moment Sums	Length	Wheel	Load	Load Sums	Moment Sums
0	w. 1	12.5	12.5	00.00	50	.....	.....	.....	5120.00
1	.....	.....	.....	12.50	51	.....	.....	.....	5312.50
2	.....	.....	.....	25.00	52	.....	.....	.....	5505.00
3	.....	.....	.....	37.50	53	.....	.....	.....	5697.50
4	.....	.....	.....	50.00	54	.....	.....	.....	5890.00
5	.....	.....	.....	62.50	55	.....	.....	.....	6082.50
6	.....	.....	.....	75.00	56	w. 10	12.5	205.0	6275.00
7	.....	.....	.....	87.50	57	.....	.....	.....	6480.00
8	w. 2	27.5	40.0	100.00	58	.....	.....	.....	6685.00
9	.....	.....	.....	140.00	59	.....	.....	.....	6890.00
10	.....	.....	.....	180.00	60	.....	.....	.....	7095.00
11	.....	.....	.....	220.00	61	.....	.....	.....	7300.00
12	.....	.....	.....	260.00	62	.....	.....	.....	7505.00
13	w. 3	27.5	67.5	300.00	63	.....	.....	.....	7710.00
14	.....	.....	.....	367.50	64	w. 11	27.5	232.5	7915.00
15	.....	.....	.....	435.00	65	.....	.....	.....	8147.50
16	.....	.....	.....	502.50	66	.....	.....	.....	8380.00
17	.....	.....	.....	570.00	67	.....	.....	.....	8612.50
18	w. 4	27.5	95.0	637.50	68	.....	.....	.....	8845.00
19	.....	.....	.....	732.50	69	w. 12	27.5	260.0	9077.50
20	.....	.....	.....	827.50	70	.....	.....	.....	9337.50
21	.....	.....	.....	922.50	71	.....	.....	.....	9597.50
22	.....	.....	.....	1017.50	72	.....	.....	.....	9857.50
23	w. 5	27.5	122.5	1112.50	73	.....	.....	.....	10117.50
24	.....	.....	.....	1235.00	74	w. 13	27.5	287.5	10377.50
25	.....	.....	.....	1357.50	75	.....	.....	.....	10665.00
26	.....	.....	.....	1480.00	76	.....	.....	.....	10952.50
27	.....	.....	.....	1602.50	77	.....	.....	.....	11240.00
28	.....	.....	.....	1725.00	78	.....	.....	.....	11527.50
29	.....	.....	.....	1847.50	79	w. 14	27.5	315.0	11815.00
30	.....	.....	.....	1970.00	80	.....	.....	.....	12130.00
31	.....	.....	.....	2092.50	81	.....	.....	.....	12445.00
32	w. 6	17.5	140.0	2215.00	82	.....	.....	.....	12760.00
33	.....	.....	.....	2355.00	83	.....	.....	.....	13075.00
34	.....	.....	.....	2495.00	84	.....	.....	.....	13390.00
35	.....	.....	.....	2635.00	85	.....	.....	.....	13705.00
36	.....	.....	.....	2775.00	86	.....	.....	.....	14020.00
37	w. 7	17.5	157.5	2915.00	87	.....	.....	.....	14335.00
38	.....	.....	.....	3072.50	88	w. 15	17.5	332.5	14650.00
39	.....	.....	.....	3230.00	89	.....	.....	.....	14982.50
40	.....	.....	.....	3387.50	90	.....	.....	.....	15315.00
41	.....	.....	.....	3545.00	91	.....	.....	.....	15647.50
42	.....	.....	.....	3702.50	92	.....	.....	.....	15980.00
43	w. 8	17.5	175.0	3860.00	93	w. 16	17.5	350.0	16312.50
44	.....	.....	.....	4035.00	94	.....	.....	.....	16662.50
45	.....	.....	.....	4210.00	95	.....	.....	.....	17012.50
46	.....	.....	.....	4385.00	96	.....	.....	.....	17362.50
47	.....	.....	.....	4560.00	97	.....	.....	.....	17712.50
48	w. 9	17.5	192.5	4735.00	98	.....	.....	.....	18062.50
49	.....	.....	.....	4927.50	99	w. 17	17.5	367.5	18412.50
50	.....	.....	.....	5120.00	100	.....	.....	.....	18780.00

LIVE-LOAD STRESSES

81

COMMON STANDARD 100'-150'

COMMON STANDARD 150'-200'

Length	Wheel	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
100	.....	.....	.....	18780.00	150	.....	487.5	40061.25
101	.....	.....	.....	19147.50	151	.....	490.0	40550.00
102	.....	.....	.....	19515.00	152	.....	492.5	41041.25
103	.....	.....	.....	19882.50	153	.....	495.0	41535.00
104	w. 18	17.5	385.0	20250.00	154	.....	497.5	42031.25
105	.....	.....	.....	20635.00	155	.....	500.0	42530.00
106	.....	.....	.....	21020.00	156	.....	502.5	43031.25
107	.....	.....	.....	21405.00	157	.....	505.0	43535.00
108	.....	.....	.....	21790.00	158	.....	507.5	44041.25
109	.....	.....	385.0	22175.00	159	.....	510.0	44550.00
110			387.5	22561.25	160		512.5	45061.25
111			390.0	22950.00	161		515.0	45575.00
112			392.5	23341.25	162		517.5	46091.25
113			395.0	23735.00	163		520.0	46610.00
114			397.5	24131.25	164		522.5	47131.25
115			400.0	24530.00	165		525.0	47655.00
116			402.5	24931.25	166		527.5	48181.25
117			405.0	25335.00	167		530.0	48710.00
118			407.5	25741.25	168		532.5	49241.25
119			410.0	26150.00	169		535.0	49775.00
120			412.5	26561.25	170		537.5	50311.25
121			415.0	26975.00	171		540.0	50850.00
122			417.5	27391.25	172		542.5	51391.25
123			420.0	27810.00	173		545.0	51935.00
124			422.5	28231.25	174		547.5	52481.25
125			425.0	28655.00	175		550.0	53030.00
126			427.5	29081.25	176		552.5	53581.25
127			430.0	29510.00	177		555.0	54135.00
128			432.5	29941.25	178		557.5	54691.25
129			435.0	30375.00	179		560.0	55250.00
130			437.5	30811.25	180		562.5	55811.25
131			440.0	31250.00	181		565.0	56375.00
132			442.5	31691.25	182		567.5	56941.25
133			445.0	32135.00	183		570.0	57510.00
134			447.5	32581.25	184		572.5	58081.25
135			450.0	33030.00	185		575.0	58655.00
136			452.5	33481.25	186		577.5	59231.25
137			455.0	33935.00	187		580.0	59810.00
138			457.5	34391.25	188		582.5	60391.25
139			460.0	34850.00	189		585.0	60975.00
140			462.5	35311.25	190		587.5	61561.25
141			465.0	35775.00	191		590.0	62150.00
142			467.5	36241.25	192		592.5	62741.25
143			470.0	36710.00	193		595.0	63335.00
144			472.5	37181.25	194		597.5	63931.25
145			475.0	37655.00	195		600.0	64530.00
146			477.5	38131.25	196		602.5	65131.25
147			480.0	38610.00	197		605.0	65735.00
148			482.5	39091.25	198		607.5	66341.25
149			485.0	39575.00	199		610.0	66950.00
150			487.5	40061.25	200		612.5	67561.25

Uniform Load = 2,500 pounds per foot

Uniform Load = 2,500 pounds per foot

## COMMON STANDARD 200'-250'

## COMMON STANDARD 250'-300'

Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
200	Uniform Load = 2,500 pounds per foot	612.5	67561.25	250	Uniform Load = 2,500 pounds per foot	737.5	101311.25
201		615.0	68175.00	251		740.0	102050.00
202		617.5	68791.25	252		742.5	102791.25
203		620.0	69410.00	253		745.0	103535.00
204		622.5	70031.25	254		747.5	104281.25
205		625.0	70655.00	255		750.0	105030.00
206		627.5	71281.25	256		752.5	105781.25
207		630.0	71910.00	257		755.0	106535.00
208		632.5	72541.25	258		757.5	107291.25
209		635.0	73175.00	259		760.0	108050.00
210		637.5	73811.25	260		762.5	108811.25
211		640.0	74450.00	261		765.0	109575.00
212		642.5	75091.25	262		767.5	110341.25
213		645.0	75735.00	263		770.0	111110.00
214		647.5	76381.25	264		772.5	111881.25
215		650.0	77030.00	265		775.0	112655.00
216		652.5	77681.25	266		777.5	113431.25
217		655.0	78335.00	267		780.0	114210.00
218		657.5	78991.25	268		782.5	114991.25
219		660.0	79650.00	269		785.0	115775.00
220		662.5	80311.25	270		787.5	116561.25
221		665.0	80975.00	271		790.0	117350.00
222		667.5	81641.25	272		792.5	118141.25
223		670.0	82310.00	273		795.0	118935.00
224		672.5	82981.25	274		797.5	119731.25
225		675.0	83655.00	275		800.0	120530.00
226		677.5	84331.25	276		802.5	121331.25
227		680.0	85010.00	277		805.0	122135.00
228		682.5	85691.25	278		807.5	122941.25
229		685.0	86375.00	279		810.0	123750.00
230		687.5	87061.25	280		812.5	124561.25
231		690.0	87750.00	281		815.0	125375.00
232		692.5	88441.25	282		817.5	126191.25
233		695.0	89135.00	283		820.0	127010.00
234		697.5	89831.25	284		822.5	127831.25
235		700.0	90530.00	285		825.0	128655.00
236		702.5	91231.25	286		827.5	129481.25
237		705.0	91935.00	287		830.0	130310.00
238		707.5	92641.25	288		832.5	131141.25
239		710.0	93350.00	289		835.0	131975.00
240		712.5	94061.25	290		837.5	132811.25
241		715.0	94775.00	291		840.0	133650.00
242		717.5	95491.25	292		842.5	134491.25
243		720.0	96210.00	293		845.0	135335.00
244		722.5	96931.25	294		847.5	136181.25
245		725.0	97655.00	295		850.0	137030.00
246		727.5	98381.25	296		852.5	137881.25
247		730.0	99110.00	297		855.0	138735.00
248		732.5	99841.25	298		857.5	139591.25
249		735.0	100575.00	299		860.0	140450.00
250		737.5	101311.25	300		862.5	141311.25

LIVE-LOAD STRESSES

83

COMMON STANDARD 300'-350'

COMMON STANDARD 350'-400'

Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
300	Uniform Load = 2,500 pounds per foot	862.5	141311.25	350	Uniform Load = 2,500 pounds per foot	987.50	187561.25
301		865.0	142175.00	351		990.00	188550.00
302		867.5	143041.25	352		992.50	189541.25
303		870.0	143910.00	353		995.00	190535.00
304		872.5	144781.25	354		997.50	191531.25
305		875.0	145655.00	355		1000.00	192530.00
306		877.5	146531.25	356		1002.50	193531.25
307		880.0	147410.00	357		1005.00	194535.00
308		882.5	148291.25	358		1007.50	195541.25
309		885.0	149175.00	359		1010.00	196550.00
310		887.5	150061.25	360		1012.50	197561.25
311		890.0	150950.00	361		1015.00	198575.00
312		892.5	151841.25	362		1017.50	199591.25
313		895.0	152735.00	363		1020.00	200610.00
314		897.5	153631.25	364		1022.50	201631.25
315		900.0	154530.00	365		1025.00	202655.00
316		902.5	155431.25	366		1027.50	203681.25
317		905.0	156335.00	367		1030.00	204710.00
318		907.5	157241.25	368		1032.50	205741.25
319		910.0	158150.00	369		1035.00	206775.00
320		912.5	159061.25	370		1037.50	207811.25
321		915.0	159975.00	371		1040.00	208850.00
322		917.5	160891.25	372		1042.50	209891.25
323		920.0	161810.00	373		1045.00	210935.00
324		922.5	162731.25	374		1047.50	211981.25
325		925.0	163655.00	375		1050.00	213030.00
326		927.5	164581.25	376		1052.50	214081.25
327		930.0	165510.00	377		1055.00	215135.00
328		932.5	166441.25	378		1057.50	216191.25
329		935.0	167375.00	379		1060.00	217250.00
330		937.5	168311.25	380		1062.50	218311.25
331		940.0	169250.00	381		1065.00	219375.00
332		942.5	170191.25	382		1067.50	220441.25
333		945.0	171135.00	383		1070.00	221510.00
334		947.5	172081.25	384		1072.50	222581.25
335		950.0	173030.00	385		1075.00	223655.00
336		952.5	173981.25	386		1077.50	224731.25
337		955.0	174935.00	387		1080.00	225810.00
338		957.5	175891.25	388		1082.50	226891.25
339		960.0	176850.00	389		1085.00	227975.00
340		962.5	177811.25	390		1087.50	229061.25
341		965.0	178775.00	391		1090.00	230150.00
342		967.5	179741.25	392		1092.50	231241.25
343		970.0	180710.00	393		1095.00	232335.00
344		972.5	181681.25	394		1097.50	233431.25
345		975.0	182655.00	395		1100.00	234530.00
346		977.5	183631.25	396		1102.50	235631.25
347		980.0	184610.00	397		1105.00	236735.00
348		982.5	185591.25	398		1107.50	237841.25
349		985.0	186575.00	399		1110.00	238950.00
350		987.5	187561.25	400		1112.50	240061.25

## LIVE-LOAD STRESSES

LACKAWANNA 0'-50'

LACKAWANNA 50'-100'

Length	Wheel	Load	Load Sum	Moment Sum	Length	Wheel	Load	Load Sum	Moment Sum
0	w. 1	11	11.00	00.000	50	.....	..	.....	4744.000
1	.....	..	.....	11.000	51	.....	..	.....	4911.000
2	.....	..	.....	22.000	52	.....	..	.....	5078.000
3	.....	..	.....	33.000	53	.....	..	.....	5245.000
4	.....	..	.....	44.000	54	w. 10	11	178.00	5412.000
5	.....	..	.....	55.000	55	.....	..	.....	5590.000
6	.....	..	.....	66.000	56	.....	..	.....	5768.000
7	w. 2	25	36.00	77.000	57	.....	..	.....	5946.000
8	.....	..	.....	113.000	58	.....	..	.....	6124.000
9	.....	..	.....	149.000	59	.....	..	.....	6302.000
10	.....	..	.....	185.000	60	.....	..	.....	6480.000
11	.....	..	.....	221.000	61	w. 11	25	203.00	6658.000
12	w. 3	25	61.00	257.000	62	.....	..	.....	6861.000
13	.....	..	.....	318.000	63	.....	..	.....	7064.000
14	.....	..	.....	379.000	64	.....	..	.....	7267.000
15	.....	..	.....	440.000	65	.....	..	.....	7470.000
16	.....	..	.....	501.000	66	w. 12	25	228.00	7673.000
17	w. 4	25	86.00	562.000	67	.....	..	.....	7901.000
18	.....	..	.....	648.000	68	.....	..	.....	8129.000
19	.....	..	.....	734.000	69	.....	..	.....	8357.000
20	.....	..	.....	820.000	70	.....	..	.....	8585.000
21	.....	..	.....	906.000	71	w. 13	25	253.00	8813.000
22	w. 5	25	111.00	992.000	72	.....	..	.....	9066.000
23	.....	..	.....	1103.000	73	.....	..	.....	9319.000
24	.....	..	.....	1214.000	74	.....	..	.....	9572.000
25	.....	..	.....	1325.000	75	.....	..	.....	9825.000
26	.....	..	.....	1436.000	76	w. 14	25	278.00	10078.000
27	.....	..	.....	1547.000	77	.....	..	.....	10356.000
28	.....	..	.....	1658.000	78	.....	..	.....	10634.000
29	.....	..	.....	1769.000	79	.....	..	.....	10912.000
30	.....	..	.....	1880.000	80	.....	..	.....	11190.000
31	w. 6	14	125.00	1991.000	81	.....	..	.....	11468.000
32	.....	..	.....	2116.000	82	.....	..	.....	11746.000
33	.....	..	.....	2241.000	83	.....	..	.....	12024.000
34	.....	..	.....	2366.000	84	.....	..	.....	12302.000
35	.....	..	.....	2491.000	85	w. 15	14	292.00	12580.000
36	w. 7	14	139.00	2616.000	86	.....	..	.....	12872.000
37	.....	..	.....	2755.000	87	.....	..	.....	13146.000
38	.....	..	.....	2894.000	88	.....	..	.....	13456.000
39	.....	..	.....	3033.000	89	.....	..	.....	13748.000
40	.....	..	.....	3172.000	90	w. 16	14	306.00	14040.000
41	w. 8	14	153.00	3311.000	91	.....	..	.....	14346.000
42	.....	..	.....	3464.000	92	.....	..	.....	14652.000
43	.....	..	.....	3617.000	93	.....	..	.....	14958.000
44	.....	..	.....	3770.000	94	.....	..	.....	15264.000
45	.....	..	.....	3923.000	95	w. 17	14	320.00	15570.000
46	w. 9	14	167.00	4076.000	96	.....	..	.....	15890.000
47	.....	..	.....	4243.000	97	.....	..	.....	16210.000
48	.....	..	.....	4410.000	98	.....	..	.....	16530.000
49	.....	..	.....	4577.000	99	.....	..	.....	16850.000
50	.....	..	.....	4744.000	100	w. 18	14	334.00	17170.000

## LACKAWANNA 100'-150'

## LACKAWANNA 150'-200'

Length	Wheel	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
100	w. 18	14	334.00	17170.000	150		437.50	36250.500
101	.....	..	.....	17504.000	151		439.75	36689.125
102	.....	..	.....	17838.000	152		442.00	37130.000
103	.....	..	.....	18172.000	153		444.25	37573.125
104			334.00	18506.000	154		446.50	38018.500
105			336.25	18841.125	155		448.75	38466.125
106			338.50	19178.500	156		451.00	38916.000
107			340.75	19518.125	157		453.25	39368.125
108			343.00	19860.000	158		455.50	39822.500
109			345.25	20204.125	159		457.75	40279.125
110			347.50	20550.500	160		460.00	40738.000
111			349.75	20899.125	161		462.25	41199.125
112			352.00	21250.000	162		464.50	41662.500
113			354.25	21603.125	163		466.75	42128.125
114			356.50	21958.500	164		469.00	42596.000
115			358.75	22316.125	165		471.25	43066.125
116			361.00	22676.000	166		473.50	43538.500
117			363.25	23038.125	167		475.75	44013.125
118			365.50	23402.500	168		478.00	44490.000
119			367.75	23769.125	169		480.25	44969.125
120			370.00	24138.000	170		482.50	45450.500
121			372.25	24509.125	171		484.75	45934.125
122			374.50	24882.500	172		487.00	46420.000
123			376.75	25258.125	173		489.25	46908.125
124			379.00	25636.000	174		491.50	47398.500
125			381.25	26016.125	175		493.75	47891.125
126			383.50	26398.500	176		496.00	48386.000
127			385.75	26783.125	177		498.25	48883.125
128			388.00	27170.000	178		500.50	49382.500
129			390.25	27559.125	179		502.75	49884.125
130			392.50	27950.500	180		505.00	50338.000
131			394.75	28344.125	181		507.25	50894.125
132			397.00	28740.000	182		509.50	51402.500
133			399.25	29138.125	183		511.75	51913.125
134			401.50	29538.500	184		514.00	52426.000
135			403.75	29941.125	185		516.25	52941.125
136			406.00	30346.000	186		518.50	53458.500
137			408.25	30753.125	187		520.75	53978.125
138			410.50	31162.500	188		523.00	54500.000
139			412.75	31574.125	189		525.25	55024.125
140			415.00	31988.000	190		527.50	55550.500
141			417.25	32404.125	191		529.75	56079.125
142			419.50	32882.500	192		532.00	56610.000
143			421.75	33243.125	193		534.25	57143.125
144			424.00	33666.000	194		536.50	57678.500
145			426.25	34091.125	195		538.75	58216.125
146			428.50	34518.500	196		541.00	58756.000
147			430.75	34948.125	197		543.25	59298.125
148			433.00	35380.000	198		545.50	59842.500
149			435.25	35814.125	199		547.75	60389.125
150			437.50	36250.500	200		550.00	60938.000

Uniform Load = 2,250 pounds per foot

Uniform Load = 2,250 pounds per foot

## LACKAWANNA 200'-250'

## LACKAWANNA 250'-300'

Length	Load	Load Sum	Moment Sum	Length	Load	Load Sum	Moment Sum
200		550.00	60938.000	250		662.50	91250.500
201		552.25	61489.125	251		664.75	91914.125
202		554.50	62042.500	252		667.00	92580.000
203		556.75	62598.125	253		669.25	93248.125
204		559.00	63156.000	254		671.50	93918.500
205		561.25	63716.125	255		673.75	94591.125
206		563.50	64278.500	256		676.00	95266.000
207		565.75	64843.125	257		678.25	95943.125
208		568.00	65410.000	258		680.50	96622.500
209		570.25	65979.125	259		682.75	97304.125
210		572.50	66550.500	260		685.00	97988.000
211		574.75	67124.125	261		687.25	98674.125
212		577.00	67700.000	262		689.50	99362.500
213		579.25	68278.125	263		691.75	100063.125
214		581.50	68858.500	264		694.00	100746.000
215		583.75	69441.125	265		696.25	101441.125
216		586.00	70026.000	266		698.50	102138.500
217		588.25	70613.125	267		700.75	102838.125
218		590.50	71202.500	268		703.00	103540.000
219		592.75	71794.125	269		705.25	104244.125
220		595.00	72388.000	270		707.50	105950.500
221		597.25	72984.125	271		709.75	105659.125
222		599.50	73582.500	272		712.00	106370.000
223		601.75	74183.125	273		714.25	107083.125
224		604.00	74786.000	274		716.50	107798.500
225		606.25	75391.125	275		718.75	108516.125
226		608.50	75998.500	276		721.00	109236.000
227		610.75	76608.125	277		723.25	109958.125
228		613.00	77220.000	278		725.50	110682.500
229		615.25	77834.125	279		727.75	111409.125
230		617.50	78450.500	280		730.00	112138.000
231		619.75	79069.125	281		732.25	112869.125
232		622.00	79690.000	282		734.50	113602.500
233		624.25	80313.125	283		736.75	114338.125
234		626.50	80938.500	284		739.00	115076.000
235		628.75	81566.125	285		741.25	115816.125
236		631.00	82196.000	286		743.50	116558.500
237		633.25	82828.125	287		745.75	117303.125
238		635.50	83462.500	288		748.00	118050.000
239		637.75	84099.125	289		750.25	118799.125
240		640.00	84738.000	290		752.50	119550.500
241		642.25	85379.125	291		754.75	120304.125
242		644.50	86022.500	292		757.00	121060.000
243		646.75	86668.125	293		759.25	121818.125
244		649.00	87316.000	294		761.50	122578.500
245		651.25	87966.125	295		763.75	123341.125
246		653.50	88618.500	296		766.00	124106.000
247		655.75	89273.125	297		768.25	124873.125
248		658.00	89930.000	298		770.50	125642.500
249		660.25	90589.125	299		772.75	126414.125
250		662.50	91250.500	300		775.00	127188.000

Uniform Load = 2,250 pounds per foot

Uniform Load = 2,250 pounds per foot

## LIVE-LOAD STRESSES

87

LACKAWANNA 300'-350'

LACKAWANNA 350'-400'

Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
Uniform Load = 2,250 pounds per foot				Uniform Load = 2,250 pounds per foot			
300		775.00	127188.000	350		887.50	168750.500
301		777.25	127964.125	351		889.75	169639.125
302		779.50	128742.500	352		892.00	170530.000
303		781.75	129523.125	353		894.25	171423.125
304		784.00	130306.000	354		896.50	172318.500
305		786.25	131091.125	355		898.75	173216.125
306		788.50	131878.500	356		901.00	174116.000
307		790.75	132668.125	357		903.25	175018.125
308		793.00	133460.000	358		905.50	175922.500
309		795.25	134254.125	359		907.75	176829.125
310		797.50	135050.500	360		910.00	177738.000
311		799.75	135849.125	361		912.25	178649.125
312		802.00	136650.000	362		914.50	179562.500
313		804.25	137453.125	363		916.75	180478.125
314		806.50	138258.500	364		919.00	181396.000
315		808.75	139066.125	365		921.25	182316.125
316		811.00	139876.000	366		923.50	183238.500
317		813.25	140688.125	367		925.75	184163.125
318		815.50	141502.500	368		928.00	185090.000
319		817.75	142319.125	369		930.25	186019.125
320		820.00	143138.000	370		932.50	186950.500
321		822.25	143959.125	371		934.75	187884.125
322		824.50	144782.500	372		937.00	188820.000
323		826.75	145608.125	373		939.25	189758.125
324		829.00	146436.000	374		941.50	190698.500
325		831.25	147266.125	375		943.75	191641.125
326		833.50	148098.500	376		946.00	192586.000
327		835.75	148933.125	377		948.25	193533.125
328		838.00	149770.000	378		950.50	194482.500
329		840.25	150609.125	379		952.75	195434.125
330		842.50	151450.500	380		955.00	196388.000
331		844.75	152294.125	381		957.25	197344.125
332		847.00	153140.000	382		959.50	198302.500
333		849.25	153988.125	383		961.75	199263.125
334		851.50	154838.500	384		964.00	200226.000
335		853.75	155691.125	385		966.25	201191.125
336		856.00	156546.000	386		968.50	202158.500
337		858.25	157403.125	387		970.75	203128.125
338		860.50	158262.500	388		973.00	204100.000
339		862.75	159124.125	389		975.25	205074.125
340		865.00	159988.000	390		977.50	206050.500
341		867.25	160854.125	391		979.75	207029.125
342		869.50	161722.500	392		982.00	208010.000
343		871.75	162593.125	393		984.25	208993.125
344		874.00	163466.000	394		986.50	209978.500
345		876.25	164341.125	395		988.75	210966.125
346		878.50	165218.500	396		991.00	211956.000
347		880.75	166098.125	397		993.25	212948.125
348		883.00	166980.000	398		995.50	213942.500
349		885.25	167864.125	399		997.75	214939.125
350		887.50	168750.500	400		1000.00	215938.000



**TABLE 3**  
**POSITION OF COOPER'S LOADINGS FOR MAXIMUM STRESS**  
 Shorter Segment  $l_1$

Segments	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100	110	120	130	140
300-260	2	2	3	3	4	4	5	5	6	7	7	8	9	10	10	11	11	12	12	13	14	15	17	18
250-200	2	2	3	3	4	4	5	5	6	7	8	8	9	10	11	11	12	12	12	13	14	15	17	18
190-150	2	2	3	3	4	4	5	5	6	7	8	9	9	11	11	12	12	12	12	13	14	15	17	18
140	2	3	3	3	4	4	5	5	6	7	8	9	10	11	12	12	12	12	13	14	15	17	18	
130	2	3	3	3	4	4	5	5	6	7	8	9	10	11	12	12	12	12	13	14	15	17	..	
120	2	3	3	3	4	4	5	5	6	7	8	9	10	11	12	12	12	13	13	14	15	..	..	
110	2	3	3	3	4	4	5	6	7	7	8	9	10	11	12	12	12	13	13	14	..	..	..	
100	2	3	3	3	4	5	5	6	14	14	14	13	13	11	12	12	13	13	13	..	..	..	..	
95	2	3	3	4	4	5	13	13	13	13	13	13	13	13	12	12	12	13	13	..	..	..	..	
90	2	3	3	4	4	5	13	13	13	13	13	13	13	13	12	12	12	13	..	..	..	..	..	
85	2	3	3	4	4	5	13	13	12	13	13	12	13	13	12	12	12	..	..	..	..	..	..	
80	2	3	3	4	4	13	13	13	12	12	12	12	12	12	12	12	..	..	..	..	..	..	..	
75	2	3	3	4	4	13	13	12	12	12	12	12	12	12	12	..	..	..	..	..	..	..	..	
70	2	3	3	4	4	13	13	12	12	12	12	11	11	11	..	..	..	..	..	..	..	..	..	
65	2	3	3	4	4	12	12	12	12	12	11	11	11	..	..	..	..	..	..	..	..	..	..	
60	11	3	3	4	4	5	13	12	11	11	11	11	..	..	..	..	..	..	..	..	..	..	..	
55	11	12	12	12	4	12	13	12	12	13	11	..	..	..	..	..	..	..	..	..	..	..	..	
50	11	12	12	12	12	12	13	13	13	12	..	..	..	..	..	..	..	..	..	..	..	..	..	
45	2	3	12	12	12	12	13	13	13	..	..	..	..	..	..	..	..	..	..	..	..	..	..	
40	2	3	3	3	12	12	13	13	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	
35	2	3	3	4	4	13	13	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	
30	2	3	3	4	4	13	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	
25	2	3	3	4	4	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	
20	2	4	3	4	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	
15	2	3	3	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	
10	2	3	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	
5	2	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	

GENERAL NOTES.—The table gives wheel for maximum for any stress which has a triangular influence line.

In case of two unequal segments, the live load approaches on the longer segment except where wheel is overlined, when live load approaches on shorter segment.

When both segments are each greater than 142 ft., advance load on longer segment first, and upon next segment until wheel No. 1 is within 33 feet of the far end of the latter.

TABLE 4

POSITION OF COOPER'S LOADINGS FOR ABSOLUTE MAXIMUM BENDING MOMENT  
IN GIRDER BRIDGES WITHOUT PANELS

$S$  = Span in feet.  
 $c$  = Distance in feet that wheel No. 1 has moved to left beyond centre of span.  
 $w$  = wheel under which absolute maximum bending moment occurs.  
 $a$  = distance that  $w$  is to left from centre of span.  
 $b$  = " " "  $w$  " right " " " "

$S$	$c$	$w$	$a$	$b$
0' to 8'.5	8'.00	2	0'.00	....
8.5 " 11.1	9.25	2	1.25	....
11.1 " 18.7	13.00	3	0.00	....
18.7 " 27.6	14.25	3	1.25	....
27.6 " 34.9	13.39	3	0.39	....
34.9 " 38.7	17.06	4	....	0.94
38.7 " 48.6	• 18.21	4	0.21	....
48.6 " 53.7	19.45	4	1.45	....
53.7 " 58.4	74.13	13	0.13	....
58.4 " 63.2	75.37	13	1.37	....
63.2 " 70.00	74.07	13	0.07	....

NOTE.—For spans greater than 70 feet, the maximum centre moment equals the absolute maximum bending moment with an error of less than one per cent.

TABLE 5

POSITION OF COOPER'S LOADINGS FOR MAXIMUM END SHEAR IN GIRDER  
BRIDGES WITHOUT PANELS

Span	Direction Load Moves	Position of Load	Location of Maximum Shear
0' to 23'	Right to left	$w_2$ at left end	Left end
23 " 27	Right to left	$w_5$ at right end	Right end
27 " 46	Right to left	$w_2$ at left end	Left end
46 " 62	Right to left	$w_{11}$ at left end	Left end
62 " 400	Right to left	$w_2$ at left end	Left end

TABLE 6

POSITION OF COOPER'S LOADINGS FOR MAXIMUM SHEAR IN PANELS OF GIRDER  
AND TRUSS BRIDGES

Number of Panels	Panel	PANEL LENGTH IN FEET													
		22	23	24	25	26	27	28	29	30	31	32	33	34	35
6.....	0-1	4	4	4	4	4	4	4	4	4	4	5	5	5	5
	1-2	3	3	3	3	4	4	4	4	4	4	4	4	4	4
	2-3	3	3	3	3	3	3	3	3	3	3	3	3	3	4
	3-4	2	2	2	2	2	2	2	2	2	3	3	3	3	3
	4-5	2	2	2	2	2	2	2	2	2	2	2	2	2	2
7.....	0-1	4	4	4	4	4	4	4	4	4	4	4	5	5	5
	1-2	3	3	3	3	4	4	4	4	4	4	4	4	4	4
	2-3	3	3	3	3	3	3	3	3	3	3	3	3	4	4
	3-4	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	4-5	2	2	2	2	2	2	2	2	2	2	2	2	3	3
8.....	5-6	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	0-1	3	4	4	4	4	4	4	4	4	4	4	5	5	5
	1-2	3	3	3	3	4	4	4	4	4	4	4	4	4	4
	2-3	3	3	3	3	3	3	3	3	3	4	4	4	4	4
	3-4	3	3	3	3	3	3	3	3	3	3	3	3	3	3
9.....	4-5	2	2	2	2	2	3	3	3	3	3	3	3	3	3
	5-6	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	6-7	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	0-1	3	4	4	4	4	4	4	4	4	4	4	4	5	5
	1-2	3	3	3	3	4	4	4	4	4	4	4	4	4	4
10.....	2-3	3	3	3	3	3	3	3	3	4	4	4	4	4	4
	3-4	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	4-5	2	3	3	3	3	3	3	3	3	3	3	3	3	3
	5-6	2	2	2	2	2	2	2	2	2	3	3	3	3	3
	6-7	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	7-8	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	0-1	3	4	4	4	4	4	4	4	4	4	4	4	5	5
	1-2	3	3	3	3	4	4	4	4	4	4	4	4	4	4
	2-3	3	3	3	3	3	3	3	3	4	4	4	4	4	4
	3-4	3	3	3	3	3	3	3	3	3	3	3	3	3	4
	4-5	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	5-6	2	2	2	2	2	2	3	3	3	3	3	3	3	3
	6-7	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	7-8	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	8-9	1	1	1	1	1	1	1	1	2	2	2	2	2	2

NOTE.—Place tabulated wheel at right end of corresponding panel with locomotive advancing toward left.

TABLE 7

MAXIMUM MOMENTS, SHEARS, AND PIER REACTIONS FOR COOPER'S  
STANDARD LOADINGS

(Figures for One Rail)

Span	E40					E50				
	Max. Moment	Max. Shears			Max. Pier React.	Max. Moment	Max. Shears			Max. Pier React.
		End	¼ Pt.	Cent.			End	¼ Pt.	Cent.	
10.....	56.3	30.0	20.0	10.0	40.0	70.4	37.5	25.0	12.5	50.0
11.....	65.7	32.7	20.9	10.9	43.7	82.1	40.9	26.1	13.6	54.5
12.....	80.0	35.0	21.7	11.7	46.7	100.0	43.8	27.1	14.6	58.4
13.....	95.0	36.9	22.3	12.3	49.2	118.8	46.2	27.9	15.4	61.6
14.....	110.0	38.6	23.6	12.9	52.2	137.5	48.2	29.5	16.2	65.2
15.....	125.0	40.0	25.0	13.3	54.7	156.3	50.0	31.3	16.6	68.3
16.....	140.0	42.5	26.3	13.7	56.9	175.0	53.1	32.9	17.1	71.1
17.....	155.0	44.7	27.4	13.8	58.8	193.8	55.9	34.3	17.3	73.5
18.....	170.0	46.7	28.3	13.9	60.7	212.5	58.3	35.4	17.4	75.9
19.....	186.6	48.4	29.2	14.0	62.9	233.3	60.5	36.5	17.5	78.6
20.....	206.3	50.0	30.0	14.0	65.6	257.9	62.5	37.5	17.5	81.9
21.....	226.0	51.4	31.4	14.5	68.0	282.5	64.3	39.2	18.1	84.9
22.....	245.7	52.7	32.7	15.0	70.2	307.1	65.9	40.9	18.8	87.6
23.....	265.4	53.9	33.9	15.4	72.2	331.8	67.4	42.4	19.3	90.2
24.....	285.2	55.4	35.0	15.8	74.0	356.5	69.3	43.8	19.8	92.4
25.....	305.0	56.8	36.0	16.2	75.7	381.3	71.0	45.0	20.2	94.6
26.....	324.8	58.1	36.9	16.5	77.7	406.0	72.6	46.1	20.6	97.1
27.....	344.6	59.2	37.8	16.9	80.2	430.8	74.0	47.2	21.1	100.1
28.....	365.5	60.4	38.6	17.1	82.3	456.9	75.5	48.2	21.4	102.8
29.....	388.0	61.6	39.3	17.4	84.4	485.0	76.9	49.1	21.8	105.4
30.....	410.5	63.0	40.0	17.7	86.3	513.0	78.8	50.0	22.1	107.9
31.....	432.9	64.4	40.7	18.2	88.5	541.1	80.5	50.9	22.7	110.6
32.....	455.4	65.7	41.3	18.8	91.0	569.3	82.1	51.8	23.4	113.7
33.....	477.9	66.9	42.0	19.2	93.3	597.4	83.7	52.5	24.0	116.7
34.....	500.6	68.1	42.8	19.7	95.5	625.8	85.1	53.5	24.6	119.4
35.....	523.0	69.2	43.5	20.1	97.5	653.8	86.5	54.4	25.1	122.0
36.....	548.6	70.6	44.1	20.6	99.6	685.8	88.2	55.1	25.8	124.4
37.....	574.3	71.9	44.8	21.0	101.5	717.9	89.8	56.0	26.2	126.9
38.....	600.0	73.1	45.4	21.3	103.7	750.0	91.4	56.7	26.6	129.7
39.....	626.6	74.3	46.0	21.7	105.9	783.3	92.9	57.5	27.1	132.3
40.....	655.6	75.4	46.8	22.0	108.0	819.5	94.3	58.5	27.5	135.0
41.....	684.6	76.8	47.5	22.3	110.0	855.8	96.0	59.4	27.9	137.6
42.....	713.6	78.4	48.2	22.6	112.1	892.0	97.6	60.2	28.3	140.2
43.....	742.6	79.4	48.9	22.9	114.3	928.3	99.2	61.1	28.6	142.9
44.....	771.6	80.6	49.5	23.2	116.5	964.5	100.7	61.9	29.0	145.6
45.....	800.6	81.7	50.1	23.4	118.6	1000.8	102.1	62.6	29.3	148.3
46.....	829.8	82.8	50.7	23.7	120.7	1037.3	103.5	63.4	29.6	150.9
47.....	858.6	83.8	51.4	23.9	122.7	1073.3	104.9	64.2	29.9	153.4
48.....	887.6	85.0	52.1	24.2	124.8	1109.5	106.3	65.1	30.2	156.0
49.....	918.8	86.1	52.8	24.5	126.8	1148.5	107.7	66.0	30.6	158.5
50.....	950.9	87.2	53.5	24.9	128.7	1188.6	109.0	66.8	31.1	161.0
51.....	983.1	88.4	54.1	25.2	131.0	1228.9	110.4	67.6	31.5	163.6
52.....	1015.2	89.3	54.8	25.5	133.3	1269.0	111.8	68.5	31.9	166.6
53.....	1047.4	90.5	55.4	25.8	135.6	1309.2	113.1	69.2	32.3	169.6

\_\_\_\_\_

Max. Per. Home	Max. Per. School	Max. Per. Church	Max. Per. Other	Max. Per. Total
100	100	100	100	100
90	90	90	90	90
80	80	80	80	80
70	70	70	70	70
60	60	60	60	60
50	50	50	50	50
40	40	40	40	40
30	30	30	30	30
20	20	20	20	20
10	10	10	10	10
0	0	0	0	0

TABLE 7.—Continued

MAXIMUM MOMENTS, SHEARS AND PIER REACTIONS FOR COOPER'S  
STANDARD LOADINGS

(Figures for One Rail)

Span	E40					E50				
	Max. Moment	Max. Shears			Max. Pier React.	Max. Moment	Max. Shears			Max. Pier React.
		End	¼ Pt.	Cent.			End	¼ Pt.	Cent.	
98. ....	3106.5	147.5	85.0	38.8	233.6	3883.1	184.3	106.2	48.5	292.0
99. ....	3162.3	148.8	85.8	39.1	235.4	3952.9	186.0	107.2	48.9	294.2
100. ....	3219.9	150.0	86.6	39.4	237.2	4024.9	187.5	108.2	49.2	296.5
101. ....	3277.6	151.2	87.3	39.6	238.9	4097.0	189.0	109.1	49.5	298.6
102. ....	3335.9	152.4	88.1	39.9	240.6	4169.9	190.6	110.1	49.9	300.8
103. ....	3410.6	153.7	88.8	40.1	242.4	4263.3	192.1	111.0	50.1	303.0
104. ....	3475.2	154.9	89.5	40.4	244.2	4344.0	193.6	111.9	50.5	305.3
105. ....	3537.6	156.1	90.3	40.6	246.0	4422.0	195.1	112.7	50.7	307.5
106. ....	3600.3	157.3	90.9	40.9	247.8	4500.4	196.6	113.6	51.1	309.8
107. ....	3666.6	158.5	91.7	41.1	249.6	4583.3	198.1	114.5	51.5	312.0
108. ....	3745.3	159.6	92.4	41.3	251.4	4681.6	199.5	115.5	51.7	314.2
109. ....	3818.4	160.8	93.2	41.6	253.1	4773.0	201.0	116.4	52.0	316.3
110. ....	3886.8	162.0	93.9	41.8	254.8	4858.5	202.5	117.4	52.3	318.5
111. ....	3958.2	163.2	94.6	42.0	256.5	4947.7	204.0	118.2	52.5	320.7
112. ....	4026.9	164.4	95.3	42.2	258.2	5033.6	205.5	119.1	52.7	322.8
113. ....	4099.0	165.5	96.0	42.5	259.9	5123.8	207.0	120.0	53.1	324.9
114. ....	4172.0	166.7	96.8	42.8	261.6	5215.0	208.4	121.0	53.5	327.0
115. ....	4245.0	167.9	97.5	43.1	263.3	5306.2	209.9	121.9	53.9	329.0
116. ....	4318.8	169.0	98.3	43.4	264.9	5398.5	211.3	122.9	54.2	331.1
117. ....	4389.5	170.2	99.0	43.7	266.7	5486.9	212.8	123.7	54.6	333.3
118. ....	4463.8	171.4	99.7	43.9	268.5	5579.7	214.2	124.6	54.9	335.6
119. ....	4538.8	172.5	100.4	44.2	270.2	5673.5	215.7	125.5	55.3	337.8
120. ....	4614.1	173.7	101.1	44.5	272.0	5767.6	217.1	126.4	55.6	340.0
121. ....	4686.5	174.8	101.8	44.7	273.8	5858.1	218.6	127.2	55.9	342.2
122. ....	4762.7	176.0	102.5	45.0	275.6	5953.4	220.0	128.1	56.2	344.5
123. ....	4836.2	177.1	103.2	45.3	277.4	6045.2	221.4	129.0	56.5	346.7
124. ....	4917.4	178.3	104.0	45.7	279.2	6146.7	222.8	130.0	57.0	349.0
125. ....	4996.4	179.4	104.7	46.0	281.0	6245.5	224.2	130.9	57.5	351.2
150. ....	7062.3	207.4	121.8	54.4	325.4	8827.9	259.2	152.2	68.0	406.7
175. ....	9352.5	234.5	138.3	62.5	371.7	11690.6	293.1	172.9	78.2	464.6
200. ....	11873.0	261.0	153.4	70.4	419.0	14841.2	326.3	191.8	88.0	523.8
250. ....	17592.5	313.2	183.7	85.0	515.2	21990.6	391.5	229.6	106.3	644.0

NOTES.—Moments are given in thousand foot-pounds.

Shears are given in thousand pounds.

Pier reactions are given in thousand pounds and are for piers between two spans each equal to the tabulated span.

~~MAJOR MARKET OF THE WORLD—TOWNSHIP \$50 FOR ONE RAIL~~

### Aluminum: 1000 lbs. = 1000 Pounds

Project		1944		1945		1946		1947		1948		1949		1950		1951		1952		1953		1954		1955		1956		1957		1958		1959		1960	
No.	Name	No.	Name	No.	Name	No.	Name	No.	Name	No.	Name	No.	Name	No.	Name	No.	Name	No.	Name	No.	Name	No.	Name	No.	Name	No.	Name	No.	Name	No.	Name	No.	Name	No.	Name
1	...	2	...	3	...	4	...	5	...	6	...	7	...	8	...	9	...	10	...	11	...	12	...	13	...	14	...	15	...	16	...	17	...	18	...

TABLE 8.—Continued

MAXIMUM MOMENTS FOR TRUSS BRIDGES—COOPER'S E50 FOR ONE RAIL

Moments Given in Thousands of Foot-Pounds

Panel Points		0	1	2	3	4	5	6	7	8	9
Panels in Truss	Panel Points	PANEL LENGTHS									
		19' 6"	20' 0"	20' 6"	21' 0"	21' 6"	22' 0"	22' 6"	23' 0"	23' 6"	24' 0"
		24' 6"									
3	1	1404	1466	1527	1587	1653	1719	1788	1857	1927	1997
4	1	1958	2061	2166	2273	2380	2490	2597	2708	2819	2933
	2	2581	2700	2821	2946	3074	3205	3338	3471	3607	3743
5	1	2600	2731	2864	3001	3138	3279	3418	3562	3705	3852
	2	3685	3943	4144	4347	4555	4767	4978	5193	5415	5640
6	1	3210	3362	3516	3678	3840	4008	4175	4349	4522	4700
	2	4885	5256	5501	5750	5998	6250	6501	6756	7011	7270
	3	5487	5746	6028	6321	6617	6921	7228	7538	7850	8166
7	1	3778	3955	4130	4317	4505	4702	4897	5100	5303	5512
	2	6025	6326	6613	6914	7215	7530	7845	8173	8508	8842
	3	7140	7646	7990	8347	8710	9079	9448	9826	10207	10609
8	1	4320	4525	4727	4939	5150	5373	5592	5829	6061	6300
	2	7125	7458	7805	8162	8520	8890	9260	9640	10030	10430
	3	8780	9234	9330	10070	10515	10993	11475	11976	12472	12981
	4	9400	9943	10396	10862	11317	11805	12288	12790	13287	13795
9	1	4850	5179	5308	5545	5780	6030	6280	6542	6804	7074
	2	8193	8578	8970	9378	9790	10216	10640	11082	11525	11985
	3	10372	10880	11375	11900	12425	12978	13535	14118	14705	15308
	4	11605	12172	12735	13310	13880	14472	15068	15684	16300	16930

Panels in Truss	Panel Points	PANEL LENGTHS									
		25' 0"	25' 6"	26' 0"	26' 6"	27' 0"	27' 6"	28' 0"	28' 6"	29' 0"	29' 6"
		30' 0"									
3	1	2135	2215	2289	2370	2451	2534	2616	2700	2792	2889
4	1	3165	3282	3405	3526	3649	3774	3900	4031	4165	4300
	2	4025	4170	4344	4501	4681	4858	5034	5215	5398	5580
5	1	4150	4301	4456	4611	4770	4929	5092	5255	5422	5589
	2	6093	6371	6552	6783	7017	7250	7492	7736	7984	8232
6	1	5061	5245	5433	5622	5816	6010	6208	6408	6612	6817
	2	7794	8068	8352	8654	8960	9268	9580	9897	10218	10547
	3	8821	9153	9490	9828	10170	10514	10862	11208	11565	11925
7	1	5936	6151	6373	6595	6823	7051	7286	7521	7762	8003
	2	9530	9875	10236	10600	10980	11357	11742	12125	12520	12918
	3	11444	11870	12312	12752	13203	13653	14112	14571	15039	15507
8	1	6787	7035	7289	7540	7806	8069	8338	8608	8887	9165
	2	11244	11655	12080	12508	12950	13392	13850	14308	14780	15250
	3	14010	14528	15063	15605	16163	16718	17285	17852	18431	19010
	4	14820	15340	15875	16413	16965	17514	18075	18635	19210	19795
9	1	7622	7900	8188	8477	8774	9070	9376	9686	9996	10310
	2	12925	13400	13890	14380	14888	15400	15930	16460	17005	17547
	3	16528	17145	17778	18414	19070	19730	20405	21080	21770	22461
	4	18205	18850	19515	20180	20870	21567	22260	22955	23678	24405



TABLE 8.—Continued

## MAXIMUM MOMENTS FOR TRUSS BRIDGES—COOPER'S E50 FOR ONE RAIL

Moments Given in Thousands of Foot-Pounds

Panel Points		0	1	2	3	4	5	6	7	8	9		
Panels in Truss	Panel Points	PANEL LENGTHS											
		30' 6"	31' 0"	31' 6"	32' 0"	32' 6"	33' 0"	33' 6"	34' 0"	34' 6"	35' 0"	35' 6"	
3	1	3080	3175	3276	3372	3471	3570	3672	3775	3877	3978	4080	
4	1	4573	4710	4852	4994	5137	5280	5428	5576	5725	5873	5923	
	2	5957	6147	6332	6516	6715	6915	7123	7331	7536	7740	7950	
5	1	5937	6113	6295	6477	6678	6849	7039	7228	7423	7617	7814	
	2	8734	8986	9241	9496	9749	10012	10291	10590	10891	11192	11495	
6	1	7238	7450	7671	7892	8120	8347	8581	8812	9050	9288	9628	
	2	11219	11558	11903	12248	12684	12979	13354	13729	14120	14510	14902	
	3	12668	13040	13418	13796	14180	14563	14952	15341	15745	16148	16654	
7	1	8501	8752	9009	9266	9536	9806	10081	10355	10637	10919	11203	
	2	13748	14165	14590	15015	15460	15885	16358	16810	17284	17758	18234	
	3	16474	16964	17466	17968	18475	18981	19508	20015	20545	21024	21606	
8	1	9740	10030	10326	10622	10931	11239	11557	11874	12200	12526	12856	
	2	16225	16720	17227	17733	18252	18770	19311	19852	20407	20961	21518	
	3	20206	20812	21432	22051	22685	23318	23960	24601	25261	25920	26585	
	4	21022	21638	22268	22898	23549	24200	24860	25531	26216	26901	27590	
9	1	10961	11288	11625	11961	12310	12652	13018	13378	13747	14116	14490	
	2	18672	19244	19832	20419	21019	21618	22239	22860	23503	24146	24795	
	3	23886	24603	25343	26083	26839	27595	28365	29135	29923	30710	31500	
	4	25943	26715	27498	28281	29096	29910	30741	31572	32431	33290	34155	

TABLE 9  
MAXIMUM SHEARS FOR TRUSS BRIDGES—COOPER'S E50 FOR ONE RAIL  
Shears Given in Thousands of Pounds

Panels		1	2	3	4	5	6	7	8	9			
Panels in Truss	Panel	PANEL LENGTHS											
		8' 0"	8' 6"	9' 0"	9' 6"	10' 0"	10' 6"	11' 0"	11' 6"	12' 0"	12' 6"	13' 0"	13' 6"
3	1	40.6	42.1	43.5	44.8	46.4	47.9	49.1	50.4	51.6	53.0	54.3	55.9
4	2	7.3	8.0	8.8	9.5	10.0	11.0	11.8	12.5	13.2	13.7	14.3	14.9
	1	54.1	56.7	59.1	61.3	63.1	65.5	67.4	69.4	71.6	73.6	75.5	77.6
	2	23.5	25.4	27.4	28.6	30.0	31.3	32.4	33.4	34.4	35.6	36.7	37.7
5	3	2.4	3.1	3.9	4.5	5.0	5.9	6.5	7.2	7.9	8.4	8.9	9.4
	1	67.5	70.4	73.6	76.6	79.4	82.3	84.5	87.1	89.2	91.4	93.6	96.4
	2	38.8	41.0	43.0	44.9	46.7	48.7	50.3	51.9	53.8	55.5	57.1	58.7
6	3	16.3	18.0	19.5	20.8	22.0	23.1	24.0	25.0	25.9	26.9	27.8	28.7
	1	80.1	83.5	86.9	90.1	93.6	96.9	100.1	103.1	106.7	110.5	114.3	118.7
	2	52.7	55.3	57.9	60.5	62.9	65.5	67.8	70.1	72.1	74.2	76.3	78.1
7	3	30.2	33.5	34.0	35.6	37.4	39.0	40.8	41.9	43.4	44.9	46.3	47.7
	4	11.5	13.0	14.4	15.6	16.6	17.8	18.8	19.4	20.2	21.1	21.9	22.6
	1	91.1	94.6	99.2	103.4	108.0	112.8	117.5	122.9	127.5	132.0	136.5	141.4
8	2	65.5	69.1	72.4	75.3	78.4	80.9	83.9	86.1	89.0	92.0	95.0	98.8
	3	43.4	45.6	48.0	50.4	52.4	54.8	56.9	58.8	59.6	62.0	64.3	65.9
	4	24.1	26.0	27.6	29.0	30.5	32.1	33.4	34.7	36.1	37.4	38.6	39.8
9	5	8.5	9.6	10.7	11.7	12.8	13.8	14.9	15.5	16.1	16.9	17.7	18.4
	1	101.9	107.6	113.6	119.3	125.4	131.0	136.4	141.9	147.2	152.3	157.4	162.9
	2	78.2	81.7	85.2	89.1	92.5	96.0	99.8	104.1	108.4	112.6	116.7	121.0
10	3	55.8	59.0	61.9	64.5	67.4	69.6	72.3	74.4	76.8	79.5	82.2	85.0
	4	36.4	38.5	40.6	42.8	44.6	46.8	48.6	50.4	52.0	53.7	55.3	56.7
	5	19.5	21.3	22.8	24.1	25.5	26.9	28.0	29.1	30.5	31.7	32.8	33.9
	6	7.4	7.9	8.4	9.2	10.0	10.9	11.9	12.5	13.1	13.8	14.5	15.1
	1	115.2	122.3	129.2	135.6	141.9	148.4	154.5	160.8	166.4	172.0	177.6	183.5
	2	89.0	93.6	98.3	103.3	108.3	113.6	118.6	123.4	128.2	132.9	137.5	142.5
11	3	68.1	71.4	74.5	77.6	81.2	84.3	87.8	91.6	95.4	99.2	102.9	106.4
	4	48.2	51.1	53.8	56.5	58.5	60.8	63.1	65.1	67.4	69.8	72.2	74.8
	5	31.0	32.9	34.9	36.9	38.5	40.5	42.3	43.8	45.3	46.8	48.3	49.6
	6	16.0	17.5	19.1	20.3	21.5	22.7	23.9	25.0	26.2	27.3	28.3	29.3

Panels in Truss	Panel	PANEL LENGTHS											
		14' 0"	14' 6"	15' 0"	15' 6"	16' 0"	16' 6"	17' 0"	17' 6"	18' 0"	18' 6"	19' 0"	
3	1	57.4	58.7	60.0	61.5	63.0	64.3	65.6	66.9	68.2	69.5	70.8	
4	2	15.5	16.0	16.4	17.1	17.8	18.3	18.8	19.3	19.9	20.5	21.0	
	1	79.6	81.6	83.6	85.5	87.3	89.0	90.6	92.6	94.5	96.4	98.3	
	2	38.6	39.6	40.6	41.7	42.7	43.9	45.0	46.1	47.2	48.3	49.3	
5	3	9.8	10.3	10.7	11.2	11.7	12.2	12.7	13.1	13.5	13.9	14.3	
	1	99.2	102.3	105.4	108.6	111.8	115.1	118.3	121.5	124.6	127.5	130.4	
	2	60.3	61.9	63.4	64.8	66.2	67.7	69.1	70.8	72.4	74.0	75.6	
6	3	29.5	30.4	31.2	32.0	32.8	33.6	34.3	35.1	35.8	36.6	37.3	
	1	123.1	127.1	131.0	134.9	138.8	142.7	146.5	150.2	153.8	157.5	161.1	
	2	79.8	82.2	84.6	86.9	90.1	93.0	95.8	98.5	101.1	103.6	106.1	
7	3	49.1	50.4	51.7	52.9	54.0	55.3	56.5	57.6	58.6	59.7	60.7	
	4	23.3	24.1	24.8	25.6	26.3	27.0	27.6	28.3	28.9	29.6	30.2	
	1	146.2	150.9	155.5	160.1	164.6	169.0	173.3	177.5	181.6	185.7	189.7	
8	2	102.6	106.1	109.6	113.0	116.4	119.7	123.1	126.4	129.6	132.8	135.9	
	3	67.4	69.3	71.1	73.1	75.0	77.4	79.7	82.1	84.4	86.6	88.8	
	4	41.0	42.2	43.4	44.4	45.4	46.5	47.5	48.5	49.4	50.4	51.3	
9	5	19.0	19.7	20.3	21.0	21.6	22.2	22.8	23.4	24.0	24.6	25.1	
	1	168.4	173.6	178.8	183.8	188.7	193.6	198.4	203.1	207.8	212.5	217.1	
	2	125.3	129.5	133.7	137.8	141.8	145.7	149.5	153.2	156.9	160.5	164.1	
10	3	87.8	90.9	93.9	96.8	99.6	102.6	105.6	108.5	111.4	114.2	117.0	
	4	58.1	59.8	61.4	63.1	64.8	66.7	68.5	70.4	72.2	74.0	75.8	
	5	35.0	36.1	37.1	38.0	38.9	39.9	40.9	41.7	42.5	43.4	44.2	
11	6	15.7	16.4	17.0	17.6	18.1	18.7	19.2	19.8	20.3	20.8	21.3	
	1	189.4	195.1	200.8	206.3	211.8	217.3	222.7	228.0	233.2	238.4	243.6	
	2	147.4	152.1	156.8	161.3	165.7	170.1	174.5	178.8	183.0	187.2	191.3	
12	3	109.8	112.9	116.7	120.4	124.1	127.6	131.0	134.4	137.7	141.0	144.2	
	4	77.3	80.1	82.7	85.2	87.6	90.1	92.5	94.9	97.3	99.9	102.4	
	5	50.8	52.4	53.8	55.4	56.9	58.6	60.2	61.9	63.5	65.3	67.0	
13	6	30.3	31.4	32.3	33.1	33.9	34.8	35.7	36.5	37.2	38.0	38.7	

TABLE 9.—Continued  
 MAXIMUM SHEARS FOR TRUSS BRIDGES—COOPER'S E50 FOR ONE RAIL  
 Shears Given in Thousands of Pounds

Panels		1	2	3	4	5	6	7	8	9		
Panels in Truss	Panel	PANEL LENGTHS										
		19' 6"	20' 0"	20' 6"	21' 0"	21' 6"	22' 0"	22' 6"	23' 0"	23' 6"	24' 0"	24' 6"
		19' 6"	20' 0"	20' 6"	21' 0"	21' 6"	22' 0"	22' 6"	23' 0"	23' 6"	24' 0"	24' 6"
3	1	72.0	73.3	74.3	75.3	76.6	78.0	79.5	81.0	82.1	83.2	84.6
	2	21.5	22.0	22.4	22.9	23.5	24.0	24.3	24.6	25.1	25.5	25.9
4	1	100.7	103.0	105.6	108.2	110.7	113.2	115.5	117.7	120.0	122.2	124.4
	2	50.3	51.3	52.2	53.1	54.0	54.9	55.8	56.8	57.4	58.2	59.1
5	3	14.7	15.0	15.3	15.6	15.9	16.2	16.5	16.7	17.0	17.2	17.5
	1	133.5	136.6	139.8	142.9	146.0	149.0	152.0	154.9	157.8	160.5	163.3
6	2	77.4	79.1	80.9	82.6	84.4	86.1	88.0	89.9	91.7	93.5	95.1
	3	38.1	38.8	39.6	40.3	40.9	41.6	42.3	42.9	43.7	44.3	45.0
7	1	164.6	168.1	171.7	175.2	178.8	182.3	185.8	189.2	192.6	195.9	199.2
	2	106.6	111.0	113.6	116.0	118.5	120.8	123.2	125.4	127.9	130.1	132.4
8	3	62.1	63.5	65.1	66.6	68.2	69.6	71.3	72.9	74.5	75.9	77.4
	4	30.8	31.4	32.1	32.8	33.4	34.0	34.5	35.0	35.5	36.0	36.6
9	1	193.9	197.8	201.7	205.5	209.6	213.7	217.8	221.8	225.8	229.7	233.6
	2	139.0	142.0	145.0	147.9	150.9	153.7	156.1	159.3	162.1	164.8	167.6
10	3	91.0	93.1	95.4	97.5	99.6	101.6	103.8	105.8	107.9	109.8	111.8
	4	52.4	53.4	54.5	55.5	56.7	57.8	59.3	60.6	62.1	63.4	64.7
11	5	25.7	26.3	26.9	27.4	28.0	28.5	29.0	29.4	29.9	30.3	30.8
	1	221.7	226.3	230.8	235.2	239.8	244.3	248.9	253.4	258.0	262.5	267.1
12	2	167.7	171.3	174.8	178.2	181.7	185.0	188.4	191.7	195.1	198.3	201.7
	3	119.8	122.5	125.1	127.6	130.5	132.8	135.4	137.8	140.3	142.7	145.2
13	4	77.8	79.8	81.7	83.6	85.5	87.3	89.2	91.0	92.8	94.5	96.3
	5	45.2	46.1	47.1	48.0	49.0	49.4	51.0	52.1	53.1	54.1	55.3
14	6	21.9	22.4	22.9	23.4	23.9	24.4	24.9	25.3	25.7	26.0	26.5
	1	248.8	253.9	259.0	264.0	269.2	274.2	279.4	284.5	289.7	294.8	299.9
15	2	195.4	199.5	203.5	207.5	211.5	215.5	219.4	223.3	227.2	231.0	234.9
	3	147.4	150.6	153.8	156.9	160.0	163.0	166.0	169.0	172.0	175.0	177.9
16	4	104.9	107.3	109.7	112.0	114.3	116.6	118.9	121.1	123.4	125.5	127.8
	5	68.6	70.1	71.7	73.3	74.9	76.4	78.0	79.5	81.2	82.8	84.3
	6	39.6	40.4	41.3	42.1	43.0	43.9	44.9	45.8	46.7	47.6	48.6
Panels in Truss	Panel	PANEL LENGTHS										
		25' 0"	25' 6"	26' 0"	26' 6"	27' 0"	27' 6"	28' 0"	28' 6"	29' 0"	29' 6"	30' 0"
		25' 0"	25' 6"	26' 0"	26' 6"	27' 0"	27' 6"	28' 0"	28' 6"	29' 0"	29' 6"	30' 0"
3	1	86.0	87.0	88.0	89.5	91.0	92.2	93.5	94.7	96.0	97.8	99.7
	2	26.4	26.8	27.2	27.6	28.0	28.3	28.6	29.0	29.4	29.7	30.0
4	1	126.5	128.7	130.9	133.1	135.2	137.3	139.3	141.5	143.6	145.8	147.9
	2	59.7	60.5	61.3	62.1	62.9	63.8	64.6	65.6	66.5	67.4	68.3
5	3	17.8	18.1	18.4	18.6	18.9	19.1	19.3	19.6	19.8	20.1	20.3
	1	166.0	168.8	171.4	174.1	176.7	179.4	181.9	184.5	187.0	189.6	192.0
6	2	96.6	98.3	100.1	101.9	103.6	105.4	107.1	108.9	110.6	112.3	114.0
	3	45.5	46.3	46.9	47.7	48.3	49.0	49.6	50.5	51.3	52.1	52.8
7	1	202.5	205.8	209.0	212.2	215.4	218.6	221.8	224.9	228.0	231.1	234.2
	2	134.5	136.8	139.0	141.3	143.5	145.8	148.0	150.3	152.4	154.6	156.7
8	3	78.6	80.2	81.5	83.0	84.3	85.7	87.0	88.4	89.6	91.1	92.4
	4	37.1	37.6	38.1	38.6	39.1	39.6	40.0	40.5	41.0	41.7	42.4
9	1	237.4	241.4	245.2	249.1	252.8	256.6	260.3	264.1	267.7	271.4	275.0
	2	170.3	173.2	175.9	178.8	181.5	184.3	187.0	189.8	192.5	195.3	197.9
10	3	113.6	115.6	117.4	119.3	121.1	123.0	124.8	126.6	128.3	130.2	131.9
	4	65.8	67.1	68.3	69.6	70.8	72.0	73.1	74.3	75.4	76.7	77.8
11	5	31.3	31.8	32.1	32.6	33.0	33.5	33.8	34.3	34.6	35.1	35.6
	1	271.5	276.0	280.4	284.9	289.2	293.6	297.9	302.3	306.5	310.8	315.0
12	2	204.9	208.3	211.6	215.1	218.4	221.8	225.0	228.4	231.7	235.0	238.2
	3	147.5	150.0	152.3	154.7	157.0	159.4	161.7	164.0	166.1	168.5	170.2
13	4	98.0	99.8	101.4	103.1	104.6	106.3	107.9	109.5	111.0	112.6	114.1
	5	56.4	57.4	58.4	59.5	60.5	61.6	62.6	63.7	64.8	65.9	66.9
14	6	26.9	27.3	27.6	28.0	28.4	28.8	29.1	29.5	29.9	30.4	30.8
	1	304.9	310.0	315.0	320.1	325.0	330.0	334.9	339.9	344.7	349.7	354.5
15	2	238.8	242.8	246.7	250.6	254.5	258.5	262.4	266.3	270.2	274.0	277.8
	3	180.8	183.8	186.7	189.6	192.4	195.3	198.0	200.9	203.8	206.7	209.5
16	4	129.9	132.0	134.1	136.3	138.4	140.5	142.5	144.6	146.6	148.6	150.6
	5	85.8	87.4	88.9	90.4	91.8	93.3	94.8	96.2	97.6	99.0	100.4
	6	49.6	50.6	51.5	52.4	53.3	54.2	55.0	55.9	56.8	57.6	58.4

TABLE 9.—*Continued*  
 MAXIMUM SHEARS FOR TRUSS BRIDGES—COOPER'S E50 FOR ONE RAIL  
 Shears Given in Thousands of Pounds

Panels		1	2	3	4	5	6	7	8	9		
Panels in Truss	Panel	PANEL LENGTHS										
		30' 6"	31' 0"	31' 6"	32' 0"	32' 6"	33' 0"	33' 6"	34' 0"	34' 6"	35' 0"	35' 6"
3	1	101.1	102.6	104.6	106.6	108.1	109.6	111.5	113.4	114.8	116.2	117.6
	2	30.4	30.8	31.2	31.5	31.8	32.2	32.5	32.8	33.1	33.4	33.7
4	1	149.9	152.0	154.0	156.1	158.0	160.0	161.9	163.8	165.8	167.9	169.8
	2	69.1	70.0	71.7	73.3	74.4	75.4	76.4	77.4	78.4	79.4	80.5
5	3	20.6	20.9	21.1	21.3	21.6	22.0	22.2	22.5	22.7	23.0	23.3
	1	194.6	197.1	199.8	202.4	205.0	207.5	210.1	212.6	215.1	217.6	220.2
6	2	115.6	117.3	118.9	120.4	122.0	123.5	125.0	126.5	128.0	129.5	131.0
	3	53.6	54.3	55.1	55.9	56.7	57.4	58.3	59.1	60.0	60.8	61.7
7	1	237.3	240.3	243.5	246.6	249.8	252.9	256.0	259.1	262.3	265.4	268.5
	2	158.8	160.9	163.0	165.1	167.2	169.3	171.4	173.4	175.4	177.4	179.4
8	3	93.7	95.0	96.3	97.5	98.8	100.0	101.3	102.5	103.8	105.1	106.4
	4	43.0	43.6	44.4	45.1	45.8	46.4	47.2	47.9	48.6	49.3	50.0
9	1	278.7	282.3	286.0	289.6	293.4	297.1	300.9	304.7	308.4	312.0	315.7
	2	200.6	203.3	205.9	208.5	211.2	213.8	216.4	218.9	221.5	224.0	226.5
10	3	133.6	135.3	137.1	138.9	140.7	142.5	144.3	146.0	147.9	149.8	151.7
	4	79.0	80.1	81.3	82.4	83.5	84.5	85.6	86.6	87.7	88.7	89.8
11	5	36.1	36.5	37.0	37.5	38.0	38.5	39.2	39.9	40.5	41.0	41.6
	1	319.3	323.5	327.8	332.0	337.0	341.9	345.6	349.3	353.2	357.0	360.9
12	2	241.4	244.6	247.8	251.0	254.2	257.4	260.6	263.8	266.9	270.0	273.2
	3	172.8	175.4	177.8	180.1	182.5	184.8	187.1	189.4	191.7	193.9	196.2
13	4	115.7	117.3	118.7	120.3	121.9	123.4	124.9	126.3	127.7	129.1	130.5
	5	67.9	68.9	69.9	70.9	71.9	72.9	73.9	74.8	75.7	76.6	77.5
14	6	31.2	31.5	32.0	32.5	32.9	33.3	33.8	34.3	34.7	35.1	35.5
	1	359.4	364.2	369.1	373.9	378.7	383.5	388.5	393.5	398.4	403.3	408.3
15	2	281.6	285.4	289.2	293.0	296.8	300.5	304.3	308.0	311.8	315.5	319.2
	3	212.4	215.3	218.2	221.0	223.9	226.8	229.6	232.5	235.3	238.1	240.8
16	4	152.7	154.8	156.8	158.8	160.7	162.6	164.6	166.6	168.6	170.5	172.5
	5	101.8	103.1	104.5	105.9	107.3	108.6	110.0	111.4	112.7	114.0	115.4
17	6	59.4	60.3	61.2	62.0	62.9	63.8	64.7	65.5	66.3	67.1	67.8

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TABLE 10

MAXIMUM BENDING MOMENTS IN GIRDER BRIDGES WITHOUT FLOOR-BEAMS,  
COOPER'S E40 LOADING

Values in Thousands of Foot-Pounds per Rail

SHORTER SEGMENT  $l_1$

	5	10	15	20	25	30	35	40	45	50	55	60
250..	1534	3030	4514	5979	7411	8820	10203	11562	12916	14278	15628	16982
225..	1404	2769	4122	5455	6758	8034	9288	10515	11743	12976	14198	15422
200..	1273	2505	3727	4926	6098	7241	8364	9460	10560	11665	12759	13849
175..	1139	2236	3326	4390	5430	6438	7430	8391	9364	10339	11306	12266
160..	1053	2073	3082	4063	5022	5950	6862	7742	8638	9535	10424	11300
150..	1003	1962	2917	3843	4749	5620	6480	7304	8150	8994	9833	10664
140..	947	1851	2750	3620	4471	5287	6093	6862	7658	8450	9236	10016
130..	889	1738	2582	3394	4191	4951	5703	6417	7161	7901	8635	9363
120..	834	1625	2410	3164	3906	4608	5307	5964	6658	7345	8028	8704
110..	774	1509	2234	2930	3617	4260	4905	5514	6148	6782	7414	8038
100..	714	1390	2055	2690	3320	3910	4494	5053	5650	6234	6813	7387
95..	682	1329	1963	2566	3169	3730	4290	4864	5431	5991	6546	7096
90..	650	1264	1866	2444	3016	3550	4114	4661	5202	5734	6263	6786
85..	617	1200	1770	2314	2854	3365	3923	4442	4936	5458	5958	6449
80..	584	1134	1671	2186	2694	3200	3715	4205	4690	5171	5646	6117
75..	551	1070	1573	2054	2530	3008	3489	3964	4422	4874	5320	5761
70..	516	1003	1474	1923	2366	2805	3254	3706	4132	4553	4967	5378
65..	482	931	1367	1792	2202	2602	3019	3437	3831	4221	4608	4993
60..	453	864	1266	1649	2025	2389	2770	3155	3519	3884	4243	4597
55..	425	805	1172	1518	1856	2195	2546	2884	3214	3514	3859	.....
50..	397	750	1091	1398	1713	2023	2336	2634	2928	3219	.....	.....
45..	367	692	1005	1290	1567	1847	2136	2404	2669	.....	.....	.....
40..	335	635	918	1171	1419	1669	1921	2160	.....	.....	.....	.....
35..	302	570	819	1050	1272	1490	1707	.....	.....	.....	.....	.....
30..	270	506	721	918	1109	1294	.....	.....	.....	.....	.....	.....
25..	235	440	622	787	946	.....	.....	.....	.....	.....	.....	.....
20..	200	373	518	656	.....	.....	.....	.....	.....	.....	.....	.....
15..	150	300	410	.....	.....	.....	.....	.....	.....	.....	.....	.....
10..	100	200	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5..	50	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

For  $l_1$  and  $l_2$  each  $> 142$  ft.  $M = l_1 l_2 + 3800 \frac{l_2}{L}$

TABLE 10.—Continued

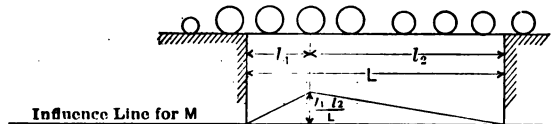
MAXIMUM BENDING MOMENTS IN GIRDER BRIDGES WITHOUT FLOOR-BEAMS,  
COOPER'S E40 LOADING

Values in Thousands of Foot-Pounds per Rail

SHORTER SEGMENT  $l_1$

	65	70	75	80	85	90	95	100	110	120	130	140
250	18327	19675	21062	22421	23766	25084	26364	27660	30152	32591	35033	37455
225	16639	17862	19123	20351	21569	22757	23908	25078	27315	29502	31691	33862
200	14939	16036	17172	18269	19360	20418	21440	22482	24465	26400	28231	30255
175	13224	14205	15207	16171	17134	18017	18952	19868	21597	23278	24963	26631
160	12185	13097	14018	14906	15789	16636	17450	18289	19866	21396	22930	24446
150	11487	12354	13194	14058	14887	15681	16442	17231	18706	20151	21569	22986
140	10790	11608	12395	13206	13980	14722	15430	16169	17542	18870	20203	21520
130	10088	10857	11594	12349	13069	13756	14413	15101	16372	17600	18834	.....
120	9380	10100	10786	11486	12073	12787	13421	14026	15197	16325	.....	.....
110	8666	9338	9972	10616	11226	11812	11392	12946	14014	.....	.....	.....
100	7963	8567	9150	9738	10294	10829	11348	11857	.....	.....	.....	.....
95	7642	8182	8737	9296	9824	10334	10834	.....	.....	.....	.....	.....
90	7303	7817	8321	8851	9352	9836	.....	.....	.....	.....	.....	.....
85	6943	7428	7917	8404	8876	.....	.....	.....	.....	.....	.....	.....
80	6582	7043	7500	7954	.....	.....	.....	.....	.....	.....	.....	.....
75	6197	6629	7057	.....	.....	.....	.....	.....	.....	.....	.....	.....
70	5796	6197	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
65	5374	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

For  $l_1$  and  $l_2$  each  $> 142$  ft.  $M = l_1 l_2 + 3800 \frac{l_1}{L}$



**TABLE 11**  
**MAXIMUM BENDING MOMENTS IN GIRDER BRIDGES WITHOUT FLOOR-BEAMS,**  
**COOPER'S, E50 LOADING**

Values in Thousands of Foot-Pounds per Rail

SHORTER SEGMENT $l_1$													
	5	10	15	20	25	30	35	40	45	50	55	60	
Longer Segment $l_2$	250.	1918	3788	5643	7474	9264	11025	12754	14452	16145	17848	19535	21228
	225.	1755	3461	5153	6819	8447	10043	11610	13144	14679	16220	17748	19278
	200.	1591	3131	4659	6158	7622	9052	10456	11825	13200	14581	15949	17311
	175.	1424	2795	4158	5487	6787	8048	9288	10489	11705	12924	14132	15333
	160.	1316	2591	3852	5079	6278	7437	8578	9677	10798	11919	13030	14125
	150.	1254	2453	3646	4804	5936	7025	8100	9130	10187	11243	12291	13330
	140.	1184	2314	3438	4525	5589	6609	7617	8578	9572	10562	11545	12520
	130.	1114	2173	3227	4242	5239	6189	7129	8021	8951	9876	10794	11704
	120.	1042	2031	3012	3955	4883	5760	6634	7455	8322	9181	10035	10880
	110.	968	1886	2793	3662	4521	5325	6131	6892	7685	8478	9268	10048
	100.	892	1737	2569	3362	4150	4887	5618	6316	7063	7793	8516	9234
	95.	853	1661	2454	3208	3961	4663	5363	6080	6789	7489	8183	8870
	90.	812	1580	2333	3055	3770	4437	5143	5826	6502	7168	7829	8482
	85.	771	1500	2213	2893	3568	4206	4904	5552	6170	6823	7448	8061
	80.	730	1418	2089	2733	3368	4000	4644	5256	5862	6464	7058	7645
	75.	689	1337	1966	2568	3163	3760	4361	4955	5528	6093	6650	7201
	70.	645	1254	1843	2404	2958	3506	4068	4632	5165	5691	6209	6721
	65.	602	1164	1709	2240	2753	3253	3774	4296	4789	5276	5760	6241
	60.	566	1080	1582	2061	2531	2986	3463	3943	4399	4855	5304	5746
	55.	531	1006	1465	1897	2320	2744	3182	3605	4017	4392	4824	5246
50.	496	937	1364	1747	2141	2529	2920	3293	3660	4024	4392	4756	
45.	459	865	1256	1613	1959	2309	2670	3005	3336	3660	3984	4308	
40.	419	794	1147	1464	1774	2086	2401	2700	2994	3284	3570	3856	
35.	377	713	1024	1312	1590	1862	2134	2394	2644	2884	3114	3344	
30.	338	632	901	1148	1386	1617	1844	2064	2274	2474	2664	2854	
25.	294	550	778	984	1182	1374	1554	1724	1884	2034	2174	2314	
20.	250	466	647	820	984	1144	1294	1434	1564	1684	1794	1904	
15.	187	375	513	647	774	894	1004	1104	1194	1274	1344	1414	
10.	125	250	375	500	625	750	875	994	1104	1204	1294	1384	
5.	62	125	187	250	312	375	437	500	562	625	687	750	

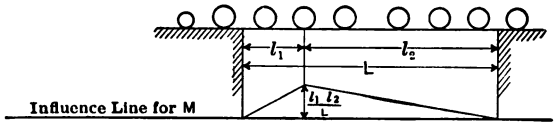
For  $l_1$  and  $l_2$  each  $> 142$  ft.  $M = 1.25 l_1 l_2 + 4750 \frac{l_2}{L}$

TABLE 11.—Continued  
MAXIMUM BENDING MOMENTS IN GIRDER BRIDGES WITHOUT FLOOR-BEAMS,  
COOPER'S E50 LOADING

Values in Thousands of Foot-Pounds per Rail

SHORTER SEGMENT $l_1$												
	65	70	75	80	85	90	95	100	110	120	130	140
250	22909	24594	26327	28026	29707	31355	32955	34575	37690	40739	43791	46819
225	20799	22327	23904	25439	26961	28446	29885	31347	34144	36878	39614	42327
200	18674	20045	21465	22836	24200	25522	26800	28102	30581	33000	35414	37819
175	16530	17756	19009	20214	21417	22521	23690	24835	26996	29098	31204	33289
160	15231	16371	17523	18633	19736	20795	21812	22861	24832	26745	28662	30558
150	14359	15443	16492	17573	18609	19601	20553	21539	23382	25189	26961	28732
140	13488	14510	15494	16508	17475	18402	19288	20211	21927	23588	25254	26900
130	12610	13571	14492	15436	16336	17195	18016	18876	20465	22000	23542	.....
120	11725	12625	13482	14357	15091	15984	16776	17533	18996	20406	.....	.....
110	10832	11672	12465	13270	14033	14765	15490	16182	17518	.....	.....	.....
100	9954	10709	11438	12173	12867	13536	14185	14821	.....	.....	.....	.....
95	9552	10227	10921	11620	12280	12917	13543	.....	.....	.....	.....	.....
90	9129	9771	10401	11064	11690	12295	.....	.....	.....	.....	.....	.....
85	8679	9285	9896	10505	11095	.....	.....	.....	.....	.....	.....	.....
80	8228	8804	9375	9943	.....	.....	.....	.....	.....	.....	.....	.....
75	7746	8286	8821	.....	.....	.....	.....	.....	.....	.....	.....	.....
70	7237	7746	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
65	6718	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

or  $l_1$  and  $l_2$  each  $> 142$  ft.  $M = 1.25 l_1 l_2 + 4750 \frac{l_1 l_2}{L}$





**TABLE 12**  
**MAXIMUM BENDING MOMENTS IN GIRDER BRIDGES WITHOUT FLOOR-BEAMS**  
**COOPER'S E60 LOADING**

Values in Thousands of Foot-pounds per Rail

SHORTER SEGMENT $l_2$												
	5	10	15	20	25	30	35	40	45	50	55	
Longer Segment $l_1$	250	2302	4547	6772	8969	11117	13230	15305	17342	19374	21418	234422
	225	2106	4153	6184	8183	10136	12052	13932	15773	17615	19464	212982
	200	1909	3757	5591	7390	9146	10862	12547	14190	15840	17497	191392
	175	1709	3354	4990	6584	8144	9658	11146	12587	14046	15509	169581
	160	1579	3109	4622	6095	7534	8924	10294	11612	12958	14303	156361
	150	1505	2944	4375	5765	7123	8430	9720	10956	12224	13492	147491
	140	1421	2777	4126	5430	6707	7931	9140	10294	11486	12674	138541
	130	1337	2608	3872	5090	6287	7427	8555	9625	10741	11851	129531
	120	1250	2437	3614	4746	5860	6912	7961	8946	9986	11017	120421
	110	1162	2263	3352	4394	5425	6390	7357	8270	9222	10174	111221
	100	1070	2084	3083	4034	4980	5864	6742	7579	8476	9352	102191
	95	1024	1993	2945	3850	4753	5596	6436	7296	8147	8987	98201
	90	974	1896	2800	3666	4524	5324	6172	6991	7802	8602	93951
	85	925	1800	2656	3472	4282	5047	5885	6662	7404	8188	8938
	80	876	1702	2507	3280	4042	4800	5573	6307	7034	7757	8470
	75	827	1604	2359	3082	3796	4512	5233	5946	6634	7312	7980
	70	774	1505	2212	2885	3550	4207	4882	5558	6198	6829	7451
	65	722	1397	2051	2688	3304	3903	4529	5155	5747	6331	6912
	60	679	1296	1898	2473	3037	3583	4156	4732	5279	5826	6365
	55	637	1207	1758	2276	2784	3293	3818	4326	4820	5270	5789
	50	595	1124	1637	2096	2569	3035	3504	3952	4392	4829	.....
	45	551	1038	1507	1936	2351	2771	3204	3606	4003	.....	.....
	40	503	953	1376	1757	2129	2503	2881	3240	.....	.....	.....
	35	452	856	1229	1574	1908	2234	2561	.....	.....	.....	.....
	30	406	758	1081	1378	1663	1940	.....	.....	.....	.....	.....
	25	353	660	934	1181	1418	.....	.....	.....	.....	.....	.....
	20	300	559	776	984	.....	.....	.....	.....	.....	.....	.....
	15	224	450	616	.....	.....	.....	.....	.....	.....	.....	.....
	10	150	300	.....	.....	.....	.....	.....	.....	.....	.....	.....
	5	74	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

For  $l_1$  and  $l_2$  each  $> 142$  ft.  $M = 1.5 \frac{l_1 l_2}{L} + 5700 \frac{l_2}{L}$

TABLE 12.—Continued

MAXIMUM BENDING MOMENTS IN GIRDER BRIDGES WITHOUT FLOOR-BEAMS,  
COOPER'S E60 LOADING

Values in Thousands of Foot-pounds per Rail

SHORTER SEGMENT $h$												
	65	70	75	80	85	90	95	100	110	120	130	140
250	27491	29513	31592	33631	35648	37626	39546	41490	45228	48887	52549	56183
225	24959	26792	28685	30527	32353	34135	35862	37616	40973	44254	47537	50792
200	22409	24054	25758	27403	29040	30626	32160	33722	36697	39600	42497	45383
175	19836	21307	22811	24257	25700	27025	28428	29802	32395	34918	37444	39947
160	18277	19645	21028	22360	23683	24954	26174	27433	29798	32094	34394	36670
150	17231	18532	19790	21088	22331	23521	24664	25847	28058	30227	32353	34478
40	16186	17412	18593	19810	20970	22082	23146	24253	26312	28306	30305	32280
130	15132	16285	17390	18523	19603	20634	21619	22651	24558	26400	28250	.....
120	14070	15150	16178	17228	18110	19181	20131	21040	22795	24487	.....	.....
110	12998	14006	14958	15924	16840	17718	18588	19418	21022	.....	.....	.....
100	11945	12851	13726	14608	15440	16243	17022	17785	.....	.....	.....	.....
95	11462	12272	13105	13944	14736	15500	16252	.....	.....	.....	.....	.....
90	10955	11725	12481	13277	14028	14754	.....	.....	.....	.....	.....	.....
85	10415	11142	11875	12606	13314	.....	.....	.....	.....	.....	.....	.....
80	9874	10565	11250	11932	.....	.....	.....	.....	.....	.....	.....	.....
75	9295	9943	10585	.....	.....	.....	.....	.....	.....	.....	.....	.....
70	8684	9295	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
65	8062	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

For  $l_1$  and  $l_2$  each  $> 142$  ft.  $M = 1.5 l_1 l_2 + 5700 \frac{l_2}{L}$

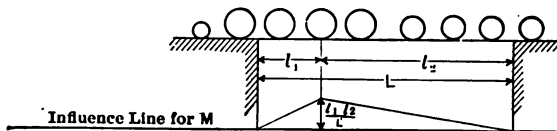


TABLE 13  
MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S  
E-40 LOADING

Values in Thousands of Pounds per Rail

		Span Ratio $\frac{L_2}{L_1}$											
		0	5	10	15	20	25	30	35	40	45	50	55
Longer Segment $L_2$	250	314	314	315	318	322	326	329	332	336	338	342	346
	225	287	287	290	294	298	301	304	306	309	312	317	321
	200	261	261	263	268	271	275	278	281	284	287	292	296
	175	234	234	236	241	244	248	251	254	258	262	266	269
	160	218	218	220	225	228	232	236	238	242	246	250	254
	150	207	207	210	214	218	221	225	229	231	234	239	244
	140	196	196	198	203	206	210	214	218	220	224	229	234
	130	185	185	187	192	196	201	203	206	210	214	219	224
	120	174	174	176	181	184	189	192	196	198	204	208	213
	110	162	162	165	170	173	178	181	185	188	193	198	202
	100	150	150	153	158	162	166	170	174	177	182	187	192
	95	144	144	146	151	155	160	163	168	173	178	182	188
	90	137	137	140	146	150	154	158	163	168	174	178	183
	85	131	131	134	139	142	148	152	158	163	168	174	178
	80	124	124	127	133	137	142	146	153	158	163	168	174
	75	118	118	122	126	130	135	140	146	152	158	162	167
	70	110	110	114	120	124	128	134	139	146	150	156	162
	65	104	104	107	112	118	122	126	133	139	144	149	155
	60	98	98	101	106	110	115	119	125	131	137	142	148
	55	93	93	95	99	103	108	113	118	125	130	134	141
	50	87	87	90	94	98	102	108	114	118	124	129	...
	45	82	82	85	90	93	98	102	109	114	118	...	...
	40	75	75	79	84	88	92	98	102	108	...	...	...
	35	69	69	74	78	82	87	92	98	...	...	...	...
	30	63	63	67	72	77	82	86	...	...	...	...	...
	25	57	57	62	66	71	76	...	...	...	...	...	...
	20	50	50	56	60	66	...	...	...	...	...	...	...
	15	40	40	50	55	...	...	...	...	...	...	...	...
	10	30	30	40	...	...	...	...	...	...	...	...	...
	5	20	20	...	...	...	...	...	...	...	...	...	...

$$\text{For } l_1 \text{ and } l_2 \text{ each } > 142 \text{ ft. } R = L + \frac{3800}{l_1}$$

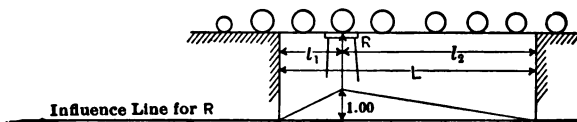
TABLE 13.—*Continued*

**MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S  
E40 LOADING**

Values in Thousands of Pounds per Rail

		SHORTER SEGMENT $l_1$												
Longer Segment $l_2$		60	65	70	75	80	85	90	95	100	110	120	130	140
	250	350	356	359	365	370	374	379	382	387	395	402	410	417
	225	326	330	334	340	345	350	354	358	362	370	377	385	392
	200	300	305	309	314	320	324	329	333	337	345	352	359	367
	175	274	279	284	290	294	300	303	308	312	319	327	334	342
	160	258	264	269	274	280	284	289	293	297	305	312	320	328
	150	248	254	259	264	269	274	278	282	287	295	302	310	318
	140	238	242	249	253	259	264	270	273	277	284	292	299	308
	130	229	233	239	243	250	254	258	262	267	274	282	290	...
	120	218	222	228	233	239	242	248	253	257	265	272	...	...
	110	207	212	218	223	230	234	238	243	247	255	...	...	...
	100	197	202	208	214	219	224	229	233	238	...	...	...	...
	95	192	198	203	208	214	219	223	229	...	...	...	...	...
	90	188	194	198	203	209	214	218	...	...	...	...	...	...
	85	183	189	194	198	204	209	...	...	...	...	...	...	...
	80	178	184	188	194	199	...	...	...	...	...	...	...	...
	75	173	178	183	188	...	...	...	...	...	...	...	...	...
	70	166	171	178	...	...	...	...	...	...	...	...	...	...
	65	160	165	...	...	...	...	...	...	...	...	...	...	...
	60	153	...	...	...	...	...	...	...	...	...	...	...	...

$$\text{For } l_1 \text{ and } l_2 \text{ each } > 142 \text{ ft. } R = L + \frac{3800}{l_1}$$



**TABLE 14**  
**MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S**  
**E50 LOADING**

Values in Thousands of Pounds per Rail

SHORTER SPAN  $l_1$

	0	5	10	15	20	25	30	35	40	45	50	55
250.....	392	392	394	398	403	407	411	415	420	423	428	432
225.....	359	359	362	367	372	376	380	383	386	390	396	401
200.....	326	326	329	335	339	344	347	351	355	359	365	370
175.....	293	293	295	301	305	310	314	318	323	327	332	336
160.....	273	273	275	281	285	290	295	298	302	307	313	318
150.....	259	259	262	267	272	277	281	286	289	293	299	305
140.....	245	245	248	254	258	263	268	273	275	280	286	293
130.....	231	231	234	240	245	251	254	260	262	268	274	280
120.....	217	217	220	226	230	236	240	245	248	255	260	266
110.....	202	202	206	212	216	222	226	231	235	241	247	253
100.....	187	187	191	197	202	208	212	218	221	227	234	240
95.....	180	180	183	189	194	200	204	210	216	222	228	235
90.....	171	171	175	182	187	192	197	204	210	218	223	229
85.....	164	164	168	174	178	185	190	198	204	210	217	223
80.....	155	155	159	166	171	177	183	191	197	204	210	217
75.....	147	147	152	158	163	169	175	183	190	197	203	209
70.....	138	138	143	150	155	160	167	174	182	188	195	202
65.....	130	130	134	140	147	152	158	166	174	180	186	194
60.....	123	123	126	132	137	144	149	156	164	171	178	185
55.....	116	116	119	124	129	135	141	148	156	162	168	176
50.....	109	109	112	118	122	128	135	142	148	155	161	...
45.....	102	102	106	112	116	122	128	136	142	148	...	...
40.....	94	94	99	105	110	115	122	128	135	...	...	...
35.....	86	86	92	98	103	109	115	122	...	...	...	...
30.....	79	79	84	90	96	102	108	...	...	...	...	...
25.....	71	71	77	83	89	95	...	...	...	...	...	...
20.....	63	63	70	75	82	...	...	...	...	...	...	...
15.....	50	50	62	69	...	...	...	...	...	...	...	...
10.....	38	38	50	...	...	...	...	...	...	...	...	...
5.....	25	25	...	...	...	...	...	...	...	...	...	...

For  $l_1$  and  $l_2$  each  $> 142$  ft.  $R = 1.25 L + \frac{4750}{l_2}$

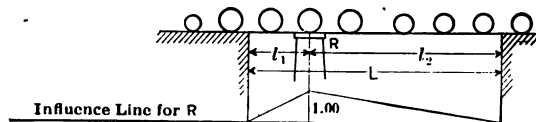
TABLE 14.—*Continued*

MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S  
E50 LOADING

Values in Thousands of Pounds per Rail

SHORTER SEGMENT $l_1$													
	60	65	70	75	80	85	90	95	100	110	120	130	140
250...	437	445	449	456	463	468	474	478	484	494	502	512	521
225...	407	413	418	425	431	437	442	448	452	462	471	481	490
200...	375	381	386	393	400	405	411	416	421	431	440	449	459
175...	343	349	355	362	368	375	379	385	390	399	409	418	427
160...	323	330	336	343	350	355	361	366	371	381	390	400	410
150...	310	317	324	330	336	343	348	353	359	369	378	387	397
140...	298	303	311	316	324	330	337	341	346	355	365	374	385
130...	286	291	299	304	312	317	323	328	334	343	352	362	...
120...	272	278	285	291	299	303	310	316	321	331	340	...	...
110...	259	265	273	279	287	292	298	304	309	319	...	...	...
100...	246	253	260	267	274	280	286	291	296	...	...	...	...
95...	240	247	254	260	267	274	279	286	...	...	...	...	...
90...	235	242	248	254	261	268	273	...	...	...	...	...	...
85...	229	236	242	248	255	261	...	...	...	...	...	...	...
80...	223	230	235	242	249	...	...	...	...	...	...	...	...
75...	216	222	229	235	...	...	...	...	...	...	...	...	...
70...	208	214	222	...	...	...	...	...	...	...	...	...	...
65...	200	206	...	...	...	...	...	...	...	...	...	...	...
60...	191	...	...	...	...	...	...	...	...	...	...	...	...

For  $l_1$  and  $l_2$  each  $> 142$  ft.  $R = 1.25 L + \frac{4750}{l_1}$



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三、中華人民共和國政府與美國政府關於台灣問題之聯合公報

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《中国书画函授大学肇庆分校建校二十周年纪念册》

本報編輯部：地址：上海南京路新報大樓

संस्कृत-विद्यायाः प्रवर्धनार्थं

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**THE UNIVERSITY OF CHICAGO**

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For  $\text{Mg} = 24.3$  and  $\text{O} = 16$  -

TABLE 15.—Continued

XIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S  
E60 LOADING

Values in Thousands of Pounds per Rail

| SHORTER SEGMENT $l_1$ |     |     |     |     |     |     |     |     |     |     |     |     |     |
|-----------------------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
|                       | 60  | 65  | 70  | 75  | 80  | 85  | 90  | 95  | 100 | 110 | 120 | 130 | 140 |
| ...                   | 524 | 534 | 539 | 547 | 556 | 562 | 569 | 574 | 581 | 593 | 602 | 614 | 625 |
| ...                   | 488 | 496 | 502 | 510 | 517 | 524 | 530 | 538 | 542 | 554 | 565 | 577 | 588 |
| ...                   | 450 | 457 | 463 | 472 | 480 | 486 | 493 | 499 | 505 | 517 | 528 | 539 | 551 |
| ...                   | 412 | 419 | 426 | 434 | 442 | 450 | 455 | 462 | 468 | 479 | 491 | 502 | 512 |
| ...                   | 388 | 396 | 403 | 412 | 420 | 426 | 433 | 439 | 445 | 457 | 468 | 480 | 492 |
| ...                   | 372 | 380 | 389 | 396 | 403 | 412 | 418 | 424 | 431 | 443 | 454 | 464 | 476 |
| ...                   | 358 | 364 | 373 | 379 | 389 | 396 | 404 | 409 | 415 | 426 | 438 | 449 | 462 |
| ...                   | 343 | 349 | 359 | 365 | 374 | 380 | 388 | 394 | 401 | 412 | 422 | 434 | ... |
| ...                   | 326 | 334 | 342 | 349 | 359 | 364 | 372 | 379 | 385 | 397 | 408 | ... | ... |
| ...                   | 311 | 318 | 328 | 335 | 344 | 350 | 358 | 365 | 371 | 383 | ... | ... | ... |
| ...                   | 295 | 304 | 312 | 320 | 329 | 336 | 343 | 349 | 356 | ... | ... | ... | ... |
| ...                   | 288 | 296 | 305 | 312 | 320 | 329 | 335 | 343 | ... | ... | ... | ... | ... |
| ...                   | 282 | 290 | 298 | 305 | 313 | 322 | 328 | ... | ... | ... | ... | ... | ... |
| ...                   | 275 | 283 | 290 | 298 | 306 | 313 | ... | ... | ... | ... | ... | ... | ... |
| ...                   | 268 | 276 | 282 | 290 | 299 | ... | ... | ... | ... | ... | ... | ... | ... |
| ...                   | 259 | 266 | 275 | 282 | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| ...                   | 250 | 257 | 266 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| ...                   | 240 | 247 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| ...                   | 229 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |

For  $l_1$  and  $l_2$  each  $>142$  ft.  $R = 1.5 L + \frac{5700}{l_1}$

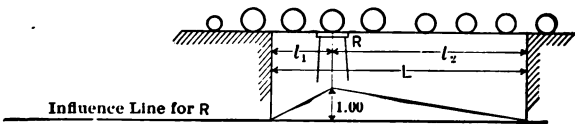




TABLE 16  
EQUIVALENT UNIFORM LOADS FOR COOPER'S E-40 LOADING

Values in Pounds per Linear Foot per Rail

|                      |     | SHORTER SEGMENT $l_1$ |      |       |       |       |       |       |       |       |       |       |       |
|----------------------|-----|-----------------------|------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
|                      |     | 0                     | 5    | 10    | 15    | 20    | 25    | 30    | 35    | 40    | 45    | 50    | 55    |
| Longer Segment $l_2$ | 250 | 2500                  | 2450 | 2430  | 2410  | 2380  | 2370  | 2350  | 2330  | 2310  | 2300  | 2290  | 2270  |
|                      | 225 | 2550                  | 2500 | 2460  | 2450  | 2430  | 2400  | 2380  | 2360  | 2340  | 2330  | 2310  | 2300  |
|                      | 200 | 2610                  | 2540 | 2500  | 2490  | 2460  | 2440  | 2420  | 2390  | 2370  | 2350  | 2340  | 2320  |
|                      | 175 | 2660                  | 2610 | 2550  | 2540  | 2510  | 2490  | 2460  | 2420  | 2400  | 2380  | 2360  | 2340  |
|                      | 150 | 2730                  | 2630 | 2560  | 2570  | 2540  | 2510  | 2480  | 2450  | 2420  | 2400  | 2380  | 2370  |
|                      | 150 | 2760                  | 2670 | 2620  | 2590  | 2570  | 2540  | 2500  | 2460  | 2430  | 2420  | 2400  | 2380  |
|                      | 140 | 2800                  | 2700 | 2650  | 2620  | 2580  | 2560  | 2520  | 2490  | 2450  | 2430  | 2420  | 2400  |
|                      | 130 | 2850                  | 2740 | 2670  | 2650  | 2610  | 2580  | 2540  | 2510  | 2470  | 2450  | 2430  | 2420  |
|                      | 120 | 2900                  | 2770 | 2710  | 2680  | 2640  | 2610  | 2560  | 2530  | 2490  | 2460  | 2450  | 2430  |
|                      | 110 | 2940                  | 2810 | 2740  | 2710  | 2660  | 2630  | 2580  | 2550  | 2500  | 2490  | 2460  | 2460  |
|                      | 100 | 3000                  | 2850 | 2780  | 2740  | 2690  | 2660  | 2610  | 2570  | 2530  | 2510  | 2500  | 2480  |
|                      | 95  | 3020                  | 2880 | 2800  | 2760  | 2700  | 2670  | 2620  | 2580  | 2560  | 2540  | 2520  | 2500  |
|                      | 90  | 3050                  | 2890 | 2810  | 2770  | 2720  | 2680  | 2630  | 2620  | 2590  | 2570  | 2550  | 2540  |
|                      | 85  | 3080                  | 2920 | 2820  | 2780  | 2730  | 2700  | 2640  | 2640  | 2620  | 2580  | 2570  | 2550  |
|                      | 80  | 3110                  | 2920 | 2840  | 2790  | 2740  | 2710  | 2670  | 2660  | 2620  | 2610  | 2580  | 2570  |
|                      | 75  | 3140                  | 2940 | 2860  | 2800  | 2740  | 2700  | 2670  | 2660  | 2640  | 2620  | 2600  | 2580  |
|                      | 70  | 3180                  | 2940 | 2870  | 2810  | 2750  | 2700  | 2670  | 2660  | 2650  | 2620  | 2600  | 2580  |
|                      | 65  | 3190                  | 2960 | 2870  | 2810  | 2760  | 2700  | 2670  | 2660  | 2650  | 2620  | 2600  | 2580  |
|                      | 60  | 3270                  | 3020 | 2880  | 2820  | 2750  | 2700  | 2660  | 2640  | 2630  | 2610  | 2590  | 2580  |
|                      | 55  | 3370                  | 3090 | 2930  | 2840  | 2760  | 2700  | 2660  | 2650  | 2620  | 2600  | 2560  | 2550  |
|                      | 50  | 3490                  | 3180 | 3000  | 2910  | 2800  | 2740  | 2700  | 2670  | 2630  | 2600  | 2580  | ..... |
|                      | 45  | 3630                  | 3260 | 3080  | 2980  | 2870  | 2780  | 2740  | 2710  | 2670  | 2640  | ..... | ..... |
|                      | 40  | 3770                  | 3350 | 3180  | 3060  | 2930  | 2840  | 2780  | 2740  | 2700  | ..... | ..... | ..... |
|                      | 35  | 3960                  | 3450 | 3260  | 3120  | 3010  | 2900  | 2840  | 2790  | ..... | ..... | ..... | ..... |
|                      | 30  | 4200                  | 3610 | 3380  | 3200  | 3060  | 2960  | 2880  | ..... | ..... | ..... | ..... | ..... |
|                      | 25  | 4540                  | 3770 | 3520  | 3320  | 3150  | 3020  | ..... | ..... | ..... | ..... | ..... | ..... |
|                      | 20  | 5000                  | 4000 | 3730  | 3450  | 3280  | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
|                      | 15  | 5336                  | 4000 | 4000  | 3650  | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
|                      | 10  | 6000                  | 4000 | 4000  | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
|                      | 5   | 8000                  | 4000 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |

For  $l_1$  and  $l_2$  each > 142 ft.  $q = \left(2.0 + \frac{7600}{l_1 L}\right) 1000$

TABLE 16.—Continued  
EQUIVALENT UNIFORM LOADS FOR COOPER'S E40 LOADING

Values in Pounds per Lineal Foot per Rail

SHORTER SEGMENT  $l_1$

|               | 60   | 65   | 70   | 75   | 80   | 85   | 90   | 95   | 100  | 110  | 120  | 130  | 140  |
|---------------|------|------|------|------|------|------|------|------|------|------|------|------|------|
| 250 . . . . . | 2260 | 2260 | 2250 | 2250 | 2240 | 2230 | 2220 | 2220 | 2210 | 2200 | 2180 | 2160 | 2140 |
| 225 . . . . . | 2290 | 2280 | 2270 | 2270 | 2260 | 2260 | 2250 | 2240 | 2220 | 2220 | 2180 | 2170 | 2150 |
| 200 . . . . . | 2310 | 2300 | 2290 | 2290 | 2280 | 2280 | 2270 | 2260 | 2250 | 2230 | 2200 | 2180 | 2160 |
| 175 . . . . . | 2340 | 2320 | 2320 | 2320 | 2310 | 2300 | 2290 | 2280 | 2270 | 2240 | 2210 | 2200 | 2180 |
| 160 . . . . . | 2350 | 2340 | 2340 | 2340 | 2330 | 2320 | 2310 | 2300 | 2280 | 2260 | 2230 | 2210 | 2180 |
| 150 . . . . . | 2370 | 2350 | 2360 | 2350 | 2340 | 2340 | 2330 | 2300 | 2300 | 2270 | 2240 | 2220 | 2190 |
| 140 . . . . . | 2380 | 2380 | 2370 | 2360 | 2360 | 2350 | 2340 | 2320 | 2310 | 2280 | 2250 | 2220 | 2200 |
| 130 . . . . . | 2400 | 2390 | 2390 | 2380 | 2380 | 2370 | 2350 | 2340 | 2330 | 2290 | 2260 | 2230 | ...  |
| 120 . . . . . | 2420 | 2410 | 2410 | 2400 | 2400 | 2370 | 2370 | 2350 | 2340 | 2300 | 2270 | ...  | ...  |
| 110 . . . . . | 2440 | 2420 | 2420 | 2420 | 2420 | 2400 | 2390 | 2380 | 2350 | 2320 | ...  | ...  | ...  |
| 100 . . . . . | 2460 | 2460 | 2450 | 2440 | 2440 | 2420 | 2410 | 2390 | 2380 | ...  | ...  | ...  | ...  |
| 95 . . . . .  | 2500 | 2480 | 2460 | 2460 | 2450 | 2440 | 2420 | 2400 | ...  | ...  | ...  | ...  | ...  |
| 90 . . . . .  | 2510 | 2500 | 2480 | 2460 | 2460 | 2450 | 2430 | ...  | ...  | ...  | ...  | ...  | ...  |
| 85 . . . . .  | 2530 | 2510 | 2500 | 2490 | 2470 | 2460 | ...  | ...  | ...  | ...  | ...  | ...  | ...  |
| 80 . . . . .  | 2550 | 2540 | 2520 | 2500 | 2490 | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  |
| 75 . . . . .  | 2560 | 2540 | 2530 | 2510 | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  |
| 70 . . . . .  | 2560 | 2540 | 2530 | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  |
| 65 . . . . .  | 2560 | 2540 | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  |
| 60 . . . . .  | 2550 | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  | ...  |

For  $l_1$  and  $l_2$  each  $> 142$  ft.  $q = \left(2.0 + \frac{7600}{l_1 L}\right) 1000$

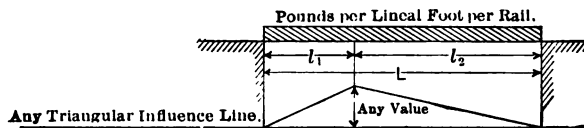


TABLE 17  
EQUIVALENT UNIFORM LOADS FOR COOPER'S E50 LOADING

Values in Pounds per Lineal Foot per Rail

SHORTER SEGMENT  $l_1$

| Longer Segment $l_2$ | SHORTER SEGMENT $l_1$ |      |       |       |       |       |       |       |       |       |       |       |
|----------------------|-----------------------|------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
|                      | 0                     | 5    | 10    | 15    | 20    | 25    | 30    | 35    | 40    | 45    | 50    | 55    |
| 250.....             | 3130                  | 3060 | 3040  | 3010  | 2980  | 2960  | 2940  | 2910  | 2890  | 2870  | 2860  | 2840  |
| 225.....             | 3190                  | 3120 | 3080  | 3060  | 3040  | 3000  | 2980  | 2950  | 2920  | 2900  | 2890  | 2870  |
| 200.....             | 3265                  | 3180 | 3130  | 3110  | 3080  | 3050  | 3020  | 2990  | 2960  | 2940  | 2920  | 2900  |
| 175.....             | 3350                  | 3260 | 3190  | 3170  | 3140  | 3110  | 3070  | 3030  | 3000  | 2970  | 2950  | 2930  |
| 160.....             | 3410                  | 3290 | 3240  | 3210  | 3170  | 3140  | 3100  | 3060  | 3020  | 3000  | 2980  | 2960  |
| 150.....             | 3455                  | 3340 | 3270  | 3240  | 3210  | 3170  | 3130  | 3080  | 3040  | 3020  | 3000  | 2980  |
| 140.....             | 3505                  | 3380 | 3305  | 3275  | 3230  | 3195  | 3150  | 3110  | 3064  | 3040  | 3018  | 3000  |
| 130.....             | 3560                  | 3420 | 3340  | 3310  | 3260  | 3225  | 3175  | 3135  | 3085  | 3060  | 3039  | 3020  |
| 120.....             | 3620                  | 3460 | 3385  | 3350  | 3295  | 3255  | 3200  | 3160  | 3106  | 3080  | 3060  | 3040  |
| 110.....             | 3680                  | 3510 | 3430  | 3385  | 3330  | 3285  | 3225  | 3185  | 3133  | 3105  | 3083  | 3065  |
| 100.....             | 3750                  | 3560 | 3470  | 3425  | 3360  | 3320  | 3260  | 3210  | 3158  | 3140  | 3117  | 3095  |
| 95.....              | 3780                  | 3600 | 3500  | 3445  | 3375  | 3340  | 3275  | 3225  | 3200  | 3175  | 3153  | 3130  |
| 90.....              | 3810                  | 3610 | 3510  | 3455  | 3395  | 3350  | 3290  | 3265  | 3237  | 3210  | 3186  | 3165  |
| 85.....              | 3850                  | 3650 | 3530  | 3470  | 3405  | 3370  | 3300  | 3295  | 3266  | 3225  | 3210  | 3185  |
| 80.....              | 3885                  | 3650 | 3545  | 3480  | 3415  | 3385  | 3335  | 3315  | 3284  | 3255  | 3232  | 3210  |
| 75.....              | 3920                  | 3670 | 3565  | 3495  | 3425  | 3380  | 3340  | 3325  | 3303  | 3275  | 3250  | 3225  |
| 70.....              | 3945                  | 3680 | 3585  | 3510  | 3435  | 3380  | 3340  | 3320  | 3308  | 3280  | 3252  | 3225  |
| 65.....              | 3990                  | 3700 | 3580  | 3505  | 3445  | 3375  | 3335  | 3325  | 3305  | 3270  | 3246  | 3220  |
| 60.....              | 4085                  | 3780 | 3595  | 3515  | 3435  | 3375  | 3315  | 3300  | 3286  | 3260  | 3237  | 3215  |
| 55.....              | 4215                  | 3860 | 3660  | 3550  | 3450  | 3380  | 3325  | 3305  | 3277  | 3245  | 3194  | 3190  |
| 50.....              | 4360                  | 3970 | 3750  | 3635  | 3495  | 3425  | 3370  | 3335  | 3293  | 3250  | 3219  | ..... |
| 45.....              | 4540                  | 4080 | 3850  | 3720  | 3585  | 3480  | 3420  | 3390  | 3339  | 3295  | ..... | ..... |
| 40.....              | 4715                  | 4190 | 3975  | 3825  | 3660  | 3550  | 3475  | 3430  | 3375  | ..... | ..... | ..... |
| 35.....              | 4945                  | 4310 | 4080  | 3900  | 3760  | 3630  | 3545  | 3485  | ..... | ..... | ..... | ..... |
| 30.....              | 5255                  | 4510 | 4215  | 4000  | 3825  | 3695  | 3595  | ..... | ..... | ..... | ..... | ..... |
| 25.....              | 5680                  | 4710 | 4400  | 4150  | 3935  | 3780  | ..... | ..... | ..... | ..... | ..... | ..... |
| 20.....              | 6250                  | 5000 | 4660  | 4315  | 4100  | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 15.....              | 6670                  | 5000 | 5000  | 4560  | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 10.....              | 7500                  | 5000 | 5000  | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 5.....               | 10000                 | 5000 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |

For  $l_1$  and  $l_2$  each  $> 142$  ft.  $q = \left( 2.5 + \frac{9500}{l_1 L} \right) 1000$

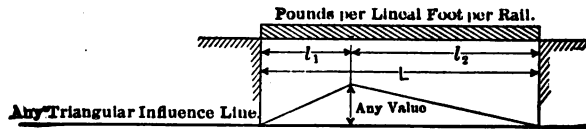
TABLE 17.—*Continued*  
EQUIVALENT UNIFORM LOADS FOR COOPER'S E50 LOADING.

Values in Pounds per Lineal Foot per Rail

SHORTER SEGMENT  $l_1$

|          | 60   | 65   | 70   | 75   | 80   | 85   | 90   | 95   | 100  | 110  | 120  | 130  | 140  |
|----------|------|------|------|------|------|------|------|------|------|------|------|------|------|
| 250..... | 2830 | 2820 | 2810 | 2810 | 2800 | 2790 | 2780 | 2770 | 2760 | 2750 | 2720 | 2700 | 2680 |
| 225..... | 2860 | 2850 | 2840 | 2840 | 2830 | 2820 | 2810 | 2800 | 2780 | 2770 | 2730 | 2710 | 2690 |
| 200..... | 2890 | 2870 | 2860 | 2860 | 2850 | 2850 | 2840 | 2820 | 2810 | 2790 | 2750 | 2720 | 2700 |
| 175..... | 2920 | 2900 | 2900 | 2900 | 2890 | 2880 | 2860 | 2850 | 2840 | 2800 | 2760 | 2750 | 2720 |
| 160..... | 2940 | 2930 | 2920 | 2920 | 2910 | 2900 | 2890 | 2870 | 2850 | 2820 | 2790 | 2760 | 2730 |
| 150..... | 2960 | 2940 | 2950 | 2940 | 2930 | 2920 | 2910 | 2880 | 2870 | 2840 | 2800 | 2770 | 2740 |
| 140..... | 2980 | 2965 | 2960 | 2950 | 2950 | 2940 | 2920 | 2900 | 2890 | 2850 | 2810 | 2775 | 2750 |
| 130..... | 3000 | 2985 | 2985 | 2975 | 2970 | 2955 | 2940 | 2920 | 2905 | 2860 | 2820 | 2785 | .... |
| 120..... | 3020 | 3005 | 3005 | 2995 | 2995 | 2960 | 2960 | 2940 | 2920 | 2880 | 2835 | .... | .... |
| 110..... | 3045 | 3030 | 3030 | 3020 | 3015 | 3000 | 2985 | 2965 | 2940 | 2895 | .... | .... | .... |
| 100..... | 3080 | 3065 | 3060 | 3050 | 3045 | 3030 | 3010 | 2985 | 2965 | .... | .... | .... | .... |
| 95.....  | 3115 | 3095 | 3075 | 3065 | 3060 | 3050 | 3020 | 3001 | .... | .... | .... | .... | .... |
| 90.....  | 3140 | 3120 | 3100 | 3080 | 3075 | 3060 | 3035 | .... | .... | .... | .... | .... | .... |
| 85.....  | 3160 | 3140 | 3120 | 3105 | 3090 | 3070 | .... | .... | .... | .... | .... | .... | .... |
| 80.....  | 3185 | 3165 | 3145 | 3125 | 3110 | .... | .... | .... | .... | .... | .... | .... | .... |
| 75.....  | 3200 | 3180 | 3155 | 3140 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 70.....  | 3200 | 3180 | 3160 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 65.....  | 3200 | 3180 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 60.....  | 3190 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |

For  $l_1$  and  $l_2$  each  $>142$  ft.  $q = \left(2.5 + \frac{9500}{l_1 L}\right) 1000$



**TABLE 18**  
**EQUIVALENT UNIFORM LOADS FOR COOPER'S E60 LOADING**

Values in Pounds per Lineal Foot per Rail

SHORTER SEGMENT  $h$

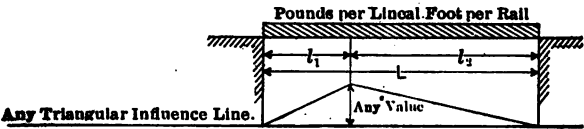
|                      |          | SHORTER SEGMENT $h$ |      |      |      |      |      |      |      |      |      |      |      |
|----------------------|----------|---------------------|------|------|------|------|------|------|------|------|------|------|------|
|                      |          | 0                   | 5    | 10   | 15   | 20   | 25   | 30   | 35   | 40   | 45   | 50   | 55   |
| Longer Segment $l_2$ | 250..... | 3760                | 3670 | 3650 | 3610 | 3580 | 3550 | 3530 | 3490 | 3470 | 3440 | 3430 | 3410 |
|                      | 225..... | 3830                | 3740 | 3700 | 3670 | 3650 | 3600 | 3580 | 3540 | 3500 | 3480 | 3470 | 3440 |
|                      | 200..... | 3920                | 3820 | 3760 | 3730 | 3700 | 3660 | 3620 | 3590 | 3550 | 3530 | 3500 | 3480 |
|                      | 175..... | 4020                | 3910 | 3830 | 3800 | 3770 | 3730 | 3680 | 3640 | 3600 | 3560 | 3540 | 3520 |
|                      | 160..... | 4090                | 3950 | 3890 | 3850 | 3800 | 3770 | 3720 | 3670 | 3620 | 3600 | 3580 | 3550 |
|                      | 150..... | 4150                | 4010 | 3920 | 3890 | 3850 | 3800 | 3760 | 3700 | 3650 | 3620 | 3600 | 3580 |
|                      | 140..... | 4210                | 4060 | 3970 | 3940 | 3880 | 3840 | 3780 | 3730 | 3680 | 3650 | 3630 | 3600 |
|                      | 130..... | 4270                | 4110 | 4010 | 3970 | 3910 | 3850 | 3820 | 3770 | 3710 | 3670 | 3650 | 3620 |
|                      | 120..... | 4340                | 4150 | 4070 | 4020 | 3960 | 3910 | 3840 | 3790 | 3730 | 3700 | 3670 | 3650 |
|                      | 110..... | 4420                | 4210 | 4120 | 4070 | 4000 | 3950 | 3880 | 3830 | 3760 | 3760 | 3700 | 3680 |
|                      | 100..... | 4500                | 4270 | 4160 | 4120 | 4030 | 3980 | 3910 | 3850 | 3790 | 3770 | 3740 | 3720 |
|                      | 95.....  | 4540                | 4320 | 4200 | 4140 | 4060 | 4010 | 3940 | 3880 | 3840 | 3820 | 3780 | 3760 |
|                      | 90.....  | 4570                | 4330 | 4210 | 4150 | 4080 | 4020 | 3950 | 3920 | 3890 | 3850 | 3830 | 3800 |
|                      | 85.....  | 4620                | 4380 | 4240 | 4160 | 4080 | 4040 | 3960 | 3960 | 3920 | 3880 | 3850 | 3830 |
|                      | 80.....  | 4660                | 4380 | 4260 | 4180 | 4100 | 4070 | 4010 | 3980 | 3940 | 3910 | 3880 | 3850 |
|                      | 75.....  | 4700                | 4400 | 4280 | 4200 | 4120 | 4060 | 4010 | 4000 | 3960 | 3940 | 3900 | 3870 |
|                      | 70.....  | 4730                | 4420 | 4310 | 4210 | 4130 | 4060 | 4010 | 3980 | 3970 | 3940 | 3900 | 3870 |
|                      | 65.....  | 4790                | 4440 | 4300 | 4210 | 4140 | 4060 | 4010 | 4000 | 3970 | 3920 | 3900 | 3860 |
|                      | 60.....  | 4900                | 4540 | 4320 | 4220 | 4130 | 4060 | 3980 | 3960 | 3950 | 3910 | 3890 | 3860 |
|                      | 55.....  | 5060                | 4630 | 4390 | 4260 | 4140 | 4060 | 4000 | 3970 | 3940 | 3900 | 3840 | 3830 |
|                      | 50.....  | 5230                | 4760 | 4500 | 4370 | 4200 | 4120 | 4040 | 4010 | 3950 | 3900 | 3860 | .... |
|                      | 45.....  | 5450                | 4900 | 4620 | 4460 | 4310 | 4180 | 4100 | 4070 | 4010 | 3960 | .... | .... |
|                      | 40.....  | 5660                | 5030 | 4780 | 4600 | 4390 | 4260 | 4180 | 4120 | 4060 | .... | .... | .... |
|                      | 35.....  | 5930                | 5170 | 4900 | 4680 | 4510 | 4360 | 4260 | 4190 | .... | .... | .... | .... |
|                      | 30.....  | 6310                | 5410 | 5060 | 4800 | 4600 | 4440 | 4320 | .... | .... | .... | .... | .... |
|                      | 25.....  | 6820                | 5650 | 5280 | 4980 | 4730 | 4540 | .... | .... | .... | .... | .... | .... |
|                      | 20.....  | 7500                | 6000 | 5590 | 5180 | 4920 | .... | .... | .... | .... | .... | .... | .... |
|                      | 15.....  | 8000                | 6000 | 6000 | 5470 | .... | .... | .... | .... | .... | .... | .... | .... |
|                      | 10.....  | 9000                | 6000 | 6000 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
|                      | 5.....   | 12000               | 6000 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |

For  $l_1$  and  $l_2$  each  $> 142$  ft.  $q = \left( 3.0 + \frac{11400}{l_1 L} \right) 1000$

TABLE 18.—Continued  
EQUIVALENT UNIFORM LOADS FOR COOPER'S E60 LOADING  
Values in Pounds per Lineal Foot per Rail

|          | SHORTER SEGMENT $l_1$ |       |       |       |       |       |       |       |       |       |       |       |       |
|----------|-----------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
|          | 60                    | 65    | 70    | 75    | 80    | 85    | 90    | 95    | 100   | 110   | 120   | 130   | 140   |
| 250..... | 3400                  | 3380  | 3370  | 3370  | 3360  | 3350  | 3340  | 3320  | 3310  | 3300  | 3260  | 3240  | 3220  |
| 225..... | 3430                  | 3420  | 3410  | 3410  | 3400  | 3380  | 3370  | 3360  | 3340  | 3320  | 3280  | 3250  | 3230  |
| 200..... | 3470                  | 3440  | 3430  | 3430  | 3420  | 3420  | 3410  | 3380  | 3370  | 3350  | 3300  | 3260  | 3240  |
| 175..... | 3500                  | 3480  | 3480  | 3480  | 3470  | 3460  | 3430  | 3420  | 3410  | 3360  | 3310  | 3300  | 3260  |
| 160..... | 3530                  | 3520  | 3500  | 3500  | 3490  | 3480  | 3470  | 3440  | 3420  | 3380  | 3350  | 3310  | 3280  |
| 150..... | 3550                  | 3530  | 3540  | 3530  | 3520  | 3500  | 3490  | 3460  | 3440  | 3410  | 3360  | 3320  | 3290  |
| 140..... | 3580                  | 3560  | 3550  | 3540  | 3540  | 3530  | 3530  | 3480  | 3470  | 3420  | 3370  | 3340  | 3300  |
| 130..... | 3600                  | 3590  | 3580  | 3570  | 3560  | 3550  | 3550  | 3500  | 3490  | 3430  | 3380  | 3350  | ..... |
| 120..... | 3620                  | 3610  | 3600  | 3590  | 3590  | 3550  | 3550  | 3530  | 3500  | 3460  | 3410  | ..... | ..... |
| 110..... | 3650                  | 3640  | 3640  | 3630  | 3620  | 3600  | 3590  | 3560  | 3530  | 3480  | ..... | ..... | ..... |
| 100..... | 3700                  | 3680  | 3670  | 3660  | 3650  | 3640  | 3610  | 3590  | 3560  | ..... | ..... | ..... | ..... |
| 95.....  | 3740                  | 3720  | 3690  | 3680  | 3670  | 3660  | 3620  | 3600  | ..... | ..... | ..... | ..... | ..... |
| 90.....  | 3770                  | 3740  | 3720  | 3700  | 3690  | 3670  | 3650  | ..... | ..... | ..... | ..... | ..... | ..... |
| 85.....  | 3790                  | 3770  | 3740  | 3730  | 3710  | 3680  | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 80.....  | 3830                  | 3800  | 3770  | 3750  | 3730  | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 75.....  | 3840                  | 3820  | 3780  | 3770  | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 70.....  | 3840                  | 3820  | 3790  | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 65.....  | 3840                  | 3820  | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 60.....  | 3830                  | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |

For  $l_1$  and  $l_2$  each  $> 142$  ft.  $q = \left( 3.0 + \frac{11400}{l_1 L} \right) 1000$



**TABLE 19**  
**INFLUENCE-LINE ORDINATES FOR  $M$  FOR GIRDER BRIDGES WITHOUT FLOOR-**  
**BEAMS**

Values of  $\frac{l_1 l_2}{L}$

### SHORTER SEGMENT 1

[illegible]

TABLE 19.—Continued

INFLUENCE-LINE ORDINATES FOR  $M$  FOR GIRDER BRIDGES WITHOUT FLOOR-BEAMSValues of  $\frac{l_1 l_2}{L}$ SHORTER SEGMENT  $l_1$ 

|          | 65   | 70   | 75   | 80   | 85   | 90   | 95   | 100  | 110  | 120  | 130  | 140  |
|----------|------|------|------|------|------|------|------|------|------|------|------|------|
| 250..... | 51.5 | 54.6 | 57.5 | 60.6 | 63.3 | 66.2 | 69.0 | 71.4 | 76.3 | 81.3 | 85.5 | 89.3 |
| 225..... | 50.5 | 53.2 | 56.2 | 58.8 | 61.7 | 64.1 | 66.7 | 69.4 | 73.5 | 78.1 | 82.0 | 86.2 |
| 200..... | 49.0 | 51.8 | 54.6 | 57.1 | 59.5 | 62.1 | 64.5 | 66.8 | 70.9 | 75.2 | 78.7 | 82.0 |
| 175..... | 47.2 | 50.0 | 52.4 | 54.9 | 57.1 | 59.5 | 61.7 | 63.7 | 67.6 | 71.4 | 74.6 | 78.1 |
| 160..... | 46.1 | 48.5 | 51.0 | 53.2 | 55.6 | 57.5 | 59.5 | 61.7 | 64.9 | 68.5 | 71.4 | 74.6 |
| 150..... | 45.2 | 47.6 | 50.0 | 52.1 | 54.3 | 56.2 | 58.1 | 59.9 | 63.3 | 66.7 | 69.4 | 72.5 |
| 140..... | 44.4 | 46.7 | 49.0 | 51.0 | 52.9 | 54.6 | 56.5 | 58.5 | 61.7 | 64.9 | 67.6 | 70.0 |
| 130..... | 43.3 | 45.5 | 47.6 | 49.5 | 51.6 | 53.2 | 55.0 | 56.5 | 59.5 | 62.5 | 65.0 | .... |
| 120..... | 42.2 | 44.3 | 46.3 | 48.1 | 49.8 | 51.5 | 53.2 | 54.6 | 57.5 | 60.0 | .... | .... |
| 110..... | 40.8 | 42.7 | 44.6 | 46.3 | 48.1 | 49.5 | 51.0 | 52.4 | 55.0 | .... | .... | .... |
| 100..... | 39.4 | 41.2 | 42.9 | 44.4 | 46.1 | 47.4 | 48.8 | 50.0 | .... | .... | .... | .... |
| 95.....  | 38.6 | 40.3 | 42.0 | 43.5 | 44.8 | 46.3 | 47.5 | .... | .... | .... | .... | .... |
| 90.....  | 37.7 | 39.4 | 41.0 | 42.4 | 43.7 | 45.0 | .... | .... | .... | .... | .... | .... |
| 85.....  | 36.8 | 38.3 | 39.8 | 41.2 | 42.5 | .... | .... | .... | .... | .... | .... | .... |
| 80.....  | 35.8 | 37.3 | 38.7 | 40.0 | .... | .... | .... | .... | .... | .... | .... | .... |
| 75.....  | 34.8 | 36.2 | 37.5 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 70.....  | 33.8 | 35.0 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 65.....  | 32.5 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |

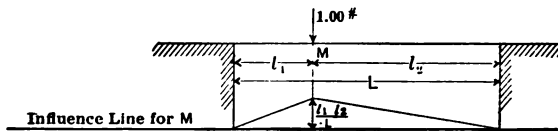






TABLE 20.—Continued  
RECIPROCAL OF INFLUENCE-LINE ORDINATES FOR *M* FOR GIRDER BRIDGES  
WITHOUT FLOOR-BEAMS

Values of  $\frac{L}{l_1 l_2}$

SHORTER SEGMENT *l*<sub>1</sub>

|     | 65    | 70    | 75    | 80    | 85    | 90    | 95    | 100   | 110   | 120   | 130   | 140   |
|-----|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| 250 | .0194 | .0183 | .0174 | .0165 | .0158 | .0151 | .0145 | .0140 | .0131 | .0123 | .0117 | .0112 |
| 225 | .0198 | .0188 | .0178 | .0170 | .0162 | .0156 | .0150 | .0144 | .0136 | .0128 | .0122 | .0116 |
| 200 | .0204 | .0193 | .0183 | .0175 | .0168 | .0161 | .0155 | .0150 | .0141 | .0133 | .0127 | .0122 |
| 175 | .0212 | .0200 | .0191 | .0182 | .0175 | .0168 | .0162 | .0157 | .0148 | .0140 | .0134 | .0128 |
| 160 | .0217 | .0206 | .0196 | .0188 | .0180 | .0174 | .0168 | .0162 | .0154 | .0146 | .0140 | .0134 |
| 150 | .0221 | .0210 | .0200 | .0192 | .0184 | .0178 | .0172 | .0167 | .0158 | .0150 | .0144 | .0138 |
| 140 | .0225 | .0214 | .0204 | .0196 | .0189 | .0183 | .0177 | .0171 | .0162 | .0154 | .0148 | .0143 |
| 130 | .0231 | .0220 | .0210 | .0202 | .0194 | .0188 | .0182 | .0177 | .0168 | .0160 | .0154 | ..... |
| 120 | .0237 | .0226 | .0216 | .0208 | .0201 | .0194 | .0188 | .0183 | .0174 | .0167 | ..... | ..... |
| 110 | .0245 | .0234 | .0224 | .0216 | .0208 | .0202 | .0196 | .0191 | .0182 | ..... | ..... | ..... |
| 100 | .0254 | .0243 | .0233 | .0225 | .0217 | .0211 | .0205 | .0200 | ..... | ..... | ..... | ..... |
| 95  | .0259 | .0248 | .0238 | .0230 | .0223 | .0216 | .0211 | ..... | ..... | ..... | ..... | ..... |
| 90  | .0265 | .0254 | .0244 | .0236 | .0229 | .0222 | ..... | ..... | ..... | ..... | ..... | ..... |
| 85  | .0272 | .0261 | .0251 | .0243 | .0235 | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 80  | .0279 | .0268 | .0258 | .0250 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 75  | .0287 | .0276 | .0266 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 70  | .0296 | .0286 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 65  | .0307 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |

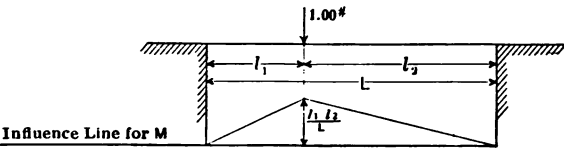




TABLE 21.—Continued

BENDING MOMENTS IN BEAMS DUE TO UNIFORM LOAD OF 1 POUND PER LINEAL FOOT

Values in Foot-pounds

Values equal  $\frac{l_1 l_2}{2}$  = Area of Influence Line for  $M$

SHORTER SEGMENT  $l_1$

|     | 65     | 70    | 75     | 80    | 85     | 90    | 95      | 100   | 110   | 120   | 130   | 140   |
|-----|--------|-------|--------|-------|--------|-------|---------|-------|-------|-------|-------|-------|
| 250 | 8125   | 8750  | 9375   | 10000 | 10625. | 11250 | 11875   | 12500 | 13750 | 15000 | 16250 | 17500 |
| 225 | 7312.5 | 7875  | 8437.5 | 9000  | 9562.5 | 10125 | 10687.5 | 11250 | 12375 | 13500 | 14625 | 15750 |
| 200 | 6500   | 7000  | 7500   | 8000  | 8500   | 9000  | 9500    | 10000 | 11000 | 12000 | 13000 | 14000 |
| 175 | 5687.5 | 6125  | 6562.5 | 7000  | 7437.5 | 7875  | 8312.5  | 8750  | 9625  | 10500 | 11375 | 12225 |
| 160 | 5200   | 5600  | 6000   | 6400  | 6800   | 7200  | 7600    | 8000  | 8800  | 9600  | 10400 | 11200 |
| 150 | 4875   | 5250  | 5625   | 6000  | 6375   | 6750  | 7125    | 7500  | 8250  | 9000  | 9750  | 10500 |
| 140 | 4550   | 4900  | 5250   | 5600  | 5950   | 6300  | 6650    | 7000  | 7700  | 8400  | 9100  | 9800  |
| 130 | 4225   | 4550  | 4875   | 5200  | 5525   | 5850  | 6175    | 6500  | 7150  | 7800  | 8450  | ..... |
| 120 | 3900   | 4200  | 4500   | 4800  | 5100   | 5400  | 5700    | 6000  | 6600  | 7200  | ..... | ..... |
| 110 | 3575   | 3850  | 4125   | 4400  | 4675   | 4950  | 5225    | 5500  | 6050  | ..... | ..... | ..... |
| 100 | 3250   | 3500  | 3750   | 4000  | 4250   | 4500  | 4750    | 5000  | ..... | ..... | ..... | ..... |
| 95  | 3087.5 | 3325  | 3562.5 | 3800  | 4037.5 | 4275  | 4512.5  | ..... | ..... | ..... | ..... | ..... |
| 90  | 2925   | 3150  | 3375   | 3600  | 3825   | 4050  | .....   | ..... | ..... | ..... | ..... | ..... |
| 85  | 2762.5 | 2975  | 3187.5 | 3400  | 3612.5 | ..... | .....   | ..... | ..... | ..... | ..... | ..... |
| 80  | 2600   | 2800  | 3000   | 3200  | .....  | ..... | .....   | ..... | ..... | ..... | ..... | ..... |
| 75  | 2437.5 | 2625  | 2812.5 | ..... | .....  | ..... | .....   | ..... | ..... | ..... | ..... | ..... |
| 70  | 2275   | 2450  | .....  | ..... | .....  | ..... | .....   | ..... | ..... | ..... | ..... | ..... |
| 65  | 2112.5 | ..... | .....  | ..... | .....  | ..... | .....   | ..... | ..... | ..... | ..... | ..... |

